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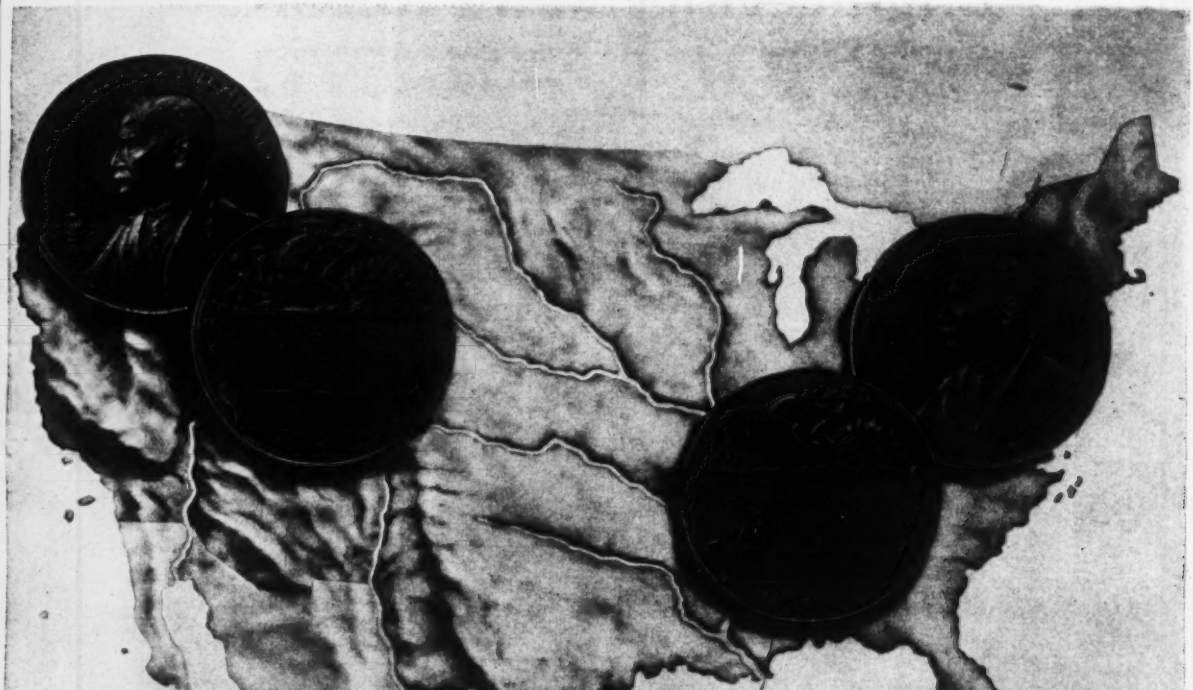

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Volume 102 NEW YORK, FEBRUARY 14, 1929 Number 7

More Power at Niagara

A TREATY providing for the construction of remedial works above the falls of Niagara is now before the Canadian Parliament and the Congress of the United States for ratification. Its primary purpose is to check the unwatering of the two flanks of the Horseshoe Falls and the drying up of the American Falls, but it includes a provision for the temporary diversion of water above the falls during the winter season in order to determine whether such additional diversions can be undertaken without detriment to the falls as a spectacle. This is a desirable combination of practical with scenic considerations. If more water can be diverted for power part of the time or all of the time without loss of scenic value, it should be done. The present plan calls for turning the additional water over temporarily to the existing power companies, in return, virtually, for their bearing the cost of construction of the remedial works. On this side of the line, where the power company is a private corporation, this proposal has caused opposition as an alienation of power which belongs to the State of New York. Such opposition should not be permitted to interfere with the work. The power available in the water so diverted is of no value except to the existing companies who have the facilities for converting it into electricity. The Niagara Falls Power Company should be allowed to use it under the agreement to protect the state's rights which it has made with the Governor.

Bascule Bridge Actions

AN ILLUMINATING addition to study of the disquieting bascule bridge failure on the Hackensack River is supplied by the designers' statement published elsewhere in this issue. It is intimated that faults either in erection or in maintenance created abnormal friction on the tail-pin bearing, and this condition is interpreted as refutation of the investigating board's charge that failure was due to structural inadequacy. But the countercharge is hardly enough to disprove the original charges. Important questions remain open. The designers' statement fails to give facts supporting the contention (made therein) that failure could not occur under normal operating conditions. What were the design assumptions for counterweight eccentricity, wind loads, friction and potential secondary actions? On what basis were the counterweight tower legs proportioned? What accounts for the buckling observed in the counterweight posts of the standing west leaf? Answers to all these questions and others are vital to the case. On the other hand, the charges of improper lubrication and inferior bronze in the bearing bushings require some explanation by the engineering board, whose report stated that there was no abnormal friction. Incidentally, the highway commission, which supervised erection and maintenance, is placed under attack. But a much more basic question grows out of the designers' statement. Suppose their claims be accepted in full: Can it be true that a bridge should collapse because one or two of its bearings were beginning to bind a little? And if it was true in the case of the Hackensack River bridge, is it sound practice to build bridges which can be wrecked and broken down by their own operating machinery? Fundamental principles of bridge engineering are at issue, and if existing codes of practice do not take account of them effectively it becomes immediately pertinent to ask whether these codes do not require revision. For this reason the failure of the Hackensack River bridge is likely to prove highly significant. The more reason, then, that its circumstances be cleared up most completely.

Better Communication

UNDER the stimulus of modern wants the world is actively calling for better means of communication. The large-scale enterprises of bridge and road building in our own midst are no doubt the outstanding example of this movement, but the world abroad gives ample evidence of the same urging. Almost overnight the pressure of the new demands has reawakened some old and long-dead projects. The Channel tunnel is again under discussion in France and England. Spain is giving new attention to the Gibraltar tunnel project. The scheme of a second Isthmian canal at Nicaragua has been brought out of dusty files. And while these old plans are being revived, an air-mail route southward across Central America is being put into operation—so rapid is the growth of interdependence and cohesiveness of the world's separated parts! We can hardly doubt that this development will lead to steadily greater demands for connection of country with country. The old projects that are just being revived may come to naught; if so, it will not be because they are before their time, but rather because newer means of transportation have taken their place. Modern needs will not be satisfied with old schemes unless they can be altered to meet present-day ideas. Methods of communication will continue to develop with changes in requirements.

Gas Tax Changes

GASOLINE tax measures are this winter agitating state legislatures to a greater degree than in several years past. It is the reaction to an impulse coming from the public and the newspapers and originating in two growing convictions—one, that gas tax rates are mounting too fast, the other, that distribution and disbursement of gas tax income should be spread to local units of governments such as counties, townships and cities and not be confined to the state. Virtually all of the bills before legislatures this year call either for an increase in gas tax rates or for a redistribution of the tax returns. The new laws proposed for the two states now without a gas tax, New York and Illinois, are bidding for support by

a definite attempt to direct the funds to the use of counties and cities as well as to the state. The objection to rate increases is more personal and individual in origin. With an average tax of 3¼ cents a gallon and maximum taxes of 5 cents, the ratio of tax to market price is beginning to appear to the automobilist as undesirably large. Altogether the situation is natural and not a cause for alarm. The gasoline tax is a recent development in imposts and in many cases represents a groping of lawmakers after a means to an end rather than action determined on understood principles and practices. Experience is gradually bringing more knowledge. The increasing agitation for readjustment comes altogether from a conviction that changed practices will better serve public roads improvement. The changes proposed, some of them decidedly wild, contain no indication of change in the belief that the gas tax is a fair and just impost for road construction and maintenance. With public opinion sound on this general principle, all problems of rates and spread of benefits will work toward a solution which will not endanger road improvement.

Stronger Structural Steel

STEELMAKING skill and technological practice have been engaged in steady advance for several decades, with the result that there is today not only cheaper and faster production but also better control of process, greater uniformity and higher quality. So far, these advances have not been capitalized in application to structural steel. It is time that this be done. A drift toward bringing out a stronger structural steel has in fact been noticeable for several years. Only the difficulties of reaching unanimity of action in a field that of necessity is highly interdependent has stood in the way. The present signs are hopeful, however, and actual realization of stronger steel appears near at hand. With greater strength, steel can of course be utilized at higher stress —quite the same advance as that made 40 years ago when steel replaced wrought iron. The working stresses and structural views of that time still prevail, despite the extensive progress at furnace and mill, but they are bound to give way as soon as steel of higher strength comes into regular production. One of the interesting questions that will be involved in this step is the relation between strength and ductility: How much elongation, for example, may fairly be demanded of a stronger steel? Possibly this relation will raise a question of another form, whether elongation is a satisfactory measure of that kind of ductility which is serviceable in structures, or whether we should return to other criteria—a bend test, for example. These questions will naturally take their place in a final decision on the adoption of a stronger steel. In the meantime a harmonizing of the remaining differences of view among steelmakers is much to be desired.

Growth and Decadence

VARIOUS signs have appeared recently that the engineering attitude of mind is receiving more general attention and understanding. Fields which by long habit have been allocated to men of business, the ministry or the law now find themselves gradually awakening to the fitness of the men of applied science, engineers, to their particular needs. Continuance of this process is likely to lead to a decrease in the sway of the legal profession in our public life, it would seem. Some dim realization of this state of affairs may be responsible for a recent large money donation to build a new law school at an Eastern university, for coupled with the gift was an expression of hope that lawyers might continue to dominate public service, in view of their special fitness. Do the gift and the statement perhaps testify to a beginning of decadence of the legal profession? Surely, in the high days of the practice of the law it needed no donations of schools to bring lawyers into the places of leadership; their own efforts elevated them to those posts. And this was true when there were proportionately fewer lawyers than now. There is, however, such a thing as professional decadence, and probably it is rather closely connected with the virility of the ideals animating the profession. Correspondingly, there may also be professional growth, similarly springing from the ideals taught in schools and in the general forums of the profession. The ideals of engineering today are high, and the growth which we believe is now becoming generally perceptible is due in large part to the strength of those ideals. It is one of our most imperative yet most easily forgotten obligations—an obligation of individual, school and society—to uphold and ever strengthen these ideals. Once they begin to lose strength, professional decadence is near.

Problem of Truck Mixers

TRUCK-MIXER concrete is becoming increasingly a factor in city building construction. It is another effort to meet those conditions surrounding construction in traffic-congested areas which have justified a high development of central mixing plants selling and delivering ready-mixed concrete. It is a development that has to be enumerated in any count of concrete merchandising practices.

Today there are perhaps fivescore commercial central mixing plants in operation, with greater or less success. With all of them the problem is delivery. To prevent segregation in transit and to maintain a workable and plastic consistency without adding water and retempering at the journey's end is the task that has caused most trouble and has been least perfectly formulated.

The truck mixer attacks the problem of delivery in merchandising concrete by making the mixing operation a function of delivery. The mixing equipment is eliminated, leaving only the proportioning equipment at the central plant, and in place of the central mixer there are substituted a few or many mixers on trucks and operated by power from the trucks.

The dry ingredients of the batch are carried in the body of the truck mixer and the water is carried separately, to be admitted when in the travel to destination it is time to throw the mixer into gear and begin to produce the merchandisable product. By proper timing it is obviously practicable by the process described to deliver a batch without segregation. Herein lies the appeal of the truck mixer as a device for the service of good concrete construction.

It is in this aspect first that concrete designers need to consider truck mixers. There are obvious economic problems, but before these can properly be considered there must be conviction of absolute surety of quality. What does the truck mixer guarantee in this respect?

There are very few precise data to tell us. Favorable opinion is plentiful, and outstanding structures of quality

can be pointed to as evidence. If, however, concrete mixture control is as important and is also as precise a procedure as the technicians contend, more assurance than this is needed that truck-mixer concrete production satisfies all requirements. In brief, test data are needed. To demand these is not condemnation of the truck mixer or of its product. It is merely requiring that these promising innovations in the field of concrete production shall submit to the same exactions of engineering proof as have older devices and their product.

A demonstrated advantage of the central mixing plant is its certainty of mixture control. In this function its possibilities are far beyond those of any but the largest and best-equipped job plants. Its drawback is that delivery of the batch has to be rigidly controlled to prevent vitiation of all that control at the mixer has contributed to quality. This control can be had by several special forms of trucks now on the market, including the agitator truck often confused with but having a quite different function than the truck mixer. The truck mixer offers a different possible corrective to delivery hazards, but it has to demonstrate its effectiveness in guaranteeing quality control of mixture. The task offers a very interesting and worth-while undertaking for concrete investigators.

More Than a Spectacle

EACH year the galaxy of exhibits shown at the Road Show reaches a higher degree of brilliance, a brilliance not only in bright colors and burnished brass but in tangible results of the equipment designer's labors. This fact, that the Road Show is much more than a spectacle, cannot be stressed too strongly. It is reflected in our report of this year's show, the concluding chapter of which was published in last week's issue.

Too often does the engineer or the contractor tend to look upon the equipment manufacturer as a mere vender of his products, when as a matter of fact he is at least in equal degree a designer—and, in this age of mechanization of construction operations, a designer of great importance to the future well-being of civil engineering activities. It is necessary to consider only a few of his contributions to appreciate this.

With water control accepted as a paramount factor in concrete manufacture, the equipment designer has provided a way to secure it by siphon and pressure tanks. Greater accuracy in aggregate measurement was demanded, and his skill produced satisfactory batchers. With speed in construction a necessity, a response was given in the motor truck, the tractor, the automatic paver and the multi-speed hoist. The increased use of the motor vehicle and its higher attainable speed makes smooth highways desirable, and the modern finishing machine as annually improved comes closer and closer to producing them.

Examples could be multiplied tenfold, but these few serve to stress the importance of the equipment designer in matters of civil engineering progress. And while he arranges a Road Show that is attractive, appealing, instructive and calculated to increase his sales, he is at the same time presenting more than an entertainment. It is his annual bid for recognition as a part of the triumvirate of which the designing engineer and the constructor are the other members. Appreciation of the Road Show in this light is a modern conception that deserves wider acceptance.

Lake Pleasant Dam

ANOTHER dam is in the limelight on account of weaknesses that menace life and property, this time a high multiple-arch in Arizona. Reckoned from lowest point of foundation to top of parapet the height of this dam is 256 ft., unprecedented for this type of structure; but even the height above streambed, 170 ft., is enough to rank it among the highest of this particular type.

It goes without saying that every structure on which depend human lives, still more so every such structure whose dimensions go beyond precedent, should be of conservative design, carefully and thoroughly checked —and it cannot be said that the Lake Pleasant dam is conservative. Its weaknesses, which fortunately came to light before there was any failure, in large measure arise from the not uncommon mistake of trying to justify a project by keeping the cost of the improvement down to the extreme minimum. More steel in the buttresses, a greater buttress length, and in general a more conservative design, would have made the structure safe for all time.

As the matter is now, with badly cracked buttresses shortening the effective base and concentrating the stresses set up by the water load, the safety of the dam is jeopardized and it is considered dangerous to allow the reservoir to fill. It is this condition that led to hurried emergency action in removing the concrete spillway and digging in its place a large outlet at lower level, as described on another page.

Whether the engineer who drew the original designs was subject to pressure from his employer is not known. In such situations the blame cannot always be placed on one individual alone. The engineer is sometimes powerless to prevent what he believes to be unsafe construction; at times, too, his advance planning, though sound, is vitiated by later developments that could not be foreseen. But at all events the plans for a dam whose stability affects the life and property of many people should be thoroughly checked by competent engineers representing the public. The State of Arizona recognized this obligation; yet it never provided funds to permit of thorough, adequate supervision and control of dams. Therein lies one major teaching of the Lake Pleasant incident.

Because of its own dereliction, Arizona is under the embarrassment of seeing subscriptions taken up to safeguard residents in the Agua Fria Valley, now believed to be menaced by a flood that would be far more destructive than that following the St. Francis dam failure. Passage of a $150,000 appropriation is being considered in the Arizona Legislature to provide for the emergency work required by Lake Pleasant dam. It is a paltry sum compared with the damage that might result if the dam failed, but it is many times more than the expenditure that would have provided adequate state supervision of dams.

California is now making a comprehensive study and a determined effort to put state supervision of dams on a sound, rational basis. It had a most impressive demonstration of the necessity for such reform less than a year ago, in the destruction wrought in the San Francisquito Valley. Many states have supervision laws of a sort, but their adequacy is not beyond doubt. New factors have entered the situation in recent years, and every state might well review its regulations and practices that affect the supervision of dam construction.

High-Speed Construction Methods at Pardee Dam

Concrete for 357-Ft. Storage Dam of Mokelumne Project Is Supplied by Four 2-Yd. Mixers in Plant Served by 4-Mile Cable Tramway—Best Month's Pour to Date 65,493 Cu.Yd.

By C. E. Grunsky, Jr.
Division Engineer, East Bay Municipal Utility District

PARDEE DAM, which is to store water on the Mokelumne River for the nine cities of the East Bay Municipal Utility District in California, had received on Feb. 1 more than 70 per cent of its total concrete yardage. The construction plant had been brought to its maximum efficiency, and for two months in succession more than 65,000 cu.yd. of concrete per month had been delivered to and placed in the structure. The plant merits study.

Pardee dam is of the curved gravity type, will be 357 ft. high, and will create a reservoir of 240,000 acre-ft. capacity to store the runoff from a 575-square mile watershed. Annual runoff at the dam site is estimated to be 900,000 acre-ft. The dam is at El. 233 and is about 85 miles east of the cities on the east side of San Francisco Bay which it is to serve. A description of the features of design of the dam appeared in *Engineering News-Record*, May 15, 1928, p. 444.

Aggregate for the concrete is obtained from gold-dredger spoilbanks located on the Mokelumne River, 4 miles downstream. Here an extensive washing and screening plant has been put in which can produce aggregate of various grades as required. Delivery to the mixing plant is made by a 4-mile cable tramway, which has operated at an average efficiency of 84.4 per cent. With careful maintenance and an operating schedule including three eight-hour shifts daily, the tramway has proved to be the limiting factor in concrete delivered to the dam. An analysis of its operation is given further on.

The total yardage of concrete in the dam is to be 615,000 cu.yd., all of which is to be in place, according to the contract, by Jan. 5, 1931. On Jan. 1 the contractor was about 350 days ahead of schedule, this advantage having been gained by a rapid rate of concrete delivery made possible by a plant layout designed for maximum speed and using four 2-cu.yd. mixers.

In addition to concrete yardage as such, the contract includes penstocks and valves incidental to the power house and to the flow of irrigation water to supply existing irrigation rights below the dam. There are four 72-in. and two 48-in. pipe lines through the dam. Two of the former will serve a power house containing two 7,500-kva. units, while the four other pipes will serve to release irrigation water.

No provision for overflow is made at the dam. A spillway is being built as a separate structure on the ridge about ¼ mile to the south. Intake to the main outlet from the reservoir is in a small gulch a mile from the dam. Here headworks will control the flow of water into the 94-mile conduit leading to distribution reservoirs in the Bay cities.

Present Status of Work—The excavation for foundation was started on Aug. 17, 1927, and between that date and Jan. 31, 1929, 142,000 cu.yd. of surface material was removed. The total estimated excavation amounts to 135,000 cu.yd.

Concreting from the main mixing plant began on Jan. 20, 1928. Before that date 107 cu.yd. was poured by using a small portable mixer. From Jan. 20 to Oct. 19, 1928, a yardage of 329,570 was poured and placed in the dam. The accompanying table indicates the progress made on this work.

CONCRETE PLACING RECORD AT PARDEE DAM

Month	Shifts	Concrete Placed, Cu.Yd. — Month's Total	Per Shift
December, 1927		107	
January, 1928	8½	3,395	388
February, 1928	33	19,808	600
March, 1928			
April, 1928	5	3,414	683
May, 1928	47½	42,064	886
June, 1928	51½	52,581	1,021
July, 1928	48½	39,620	817
August, 1928	61	65,426	1,073
September, 1928	59	65,493	1,110
October to 19th, 1928	39½	38,769	981
November, 1928	49½	45,603	920
December, 1928	58½	36,122	614
Total to Jan. 1, 1929	462	411,402	890

During the period covered by this table the mixing plant was closed down from Feb. 25 to April 26 because of flood conditions in the river.

The maximum monthly placement of concrete was

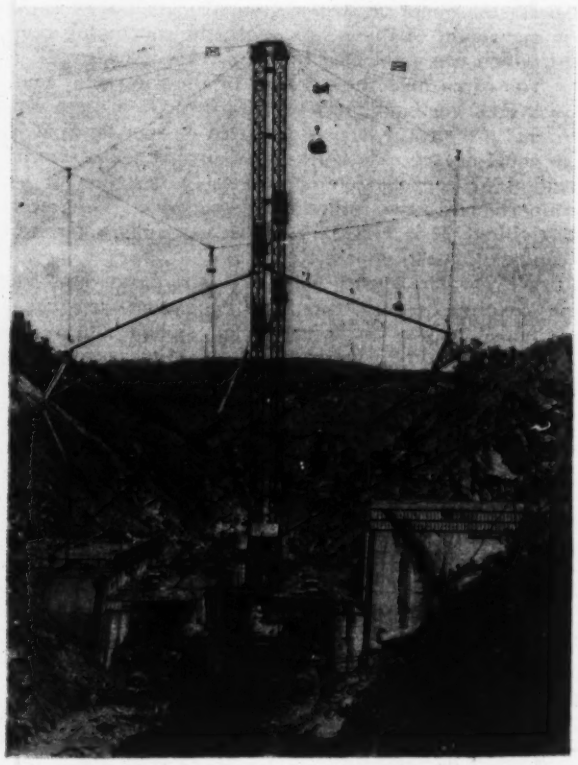

FIG. 1—PARDEE DAM AT AN EARLY STAGE
View taken June 11, 1928, showing the double tower, supply cable-way for high line at the top, and distributing chute system. The center section of the dam is kept below the side sections at all times to take flood discharge in emergencies.

achieved in September, when 65,493 cu.yd. was placed in the dam. The maximum placement during a single eight-hour shift was 1,521 cu.yd. (Aug. 24).

Gravel Plant—The gravel-washing plant is situated on the north bank of the Mokelumne River 4 miles below the dam site. At this plant the spoil resulting from early gold-digging operations on the river is excavated with a 4-yd. dragline and conveyed by an industrial railroad to the gravel plant.

By means of a 42-in. rubber conveyor belt it is lifted into the plant, where it is washed in combined screen washers, 60 in. in diameter and 21 ft. long, and sized in conical screens. This plant produces three sizes of aggregate: sand (0 to $\frac{3}{8}$ in.), gravel ($\frac{3}{8}$ to $1\frac{1}{4}$ in.), and cobbles ($1\frac{1}{4}$ to 6 in.). The material is stacked in three separate piles, providing storage of about 15,000 cu.yd.

From a tunnel under the stockpile the material is drawn and conveyed by means of a 30-in. belt to the loading terminal of the tramway.

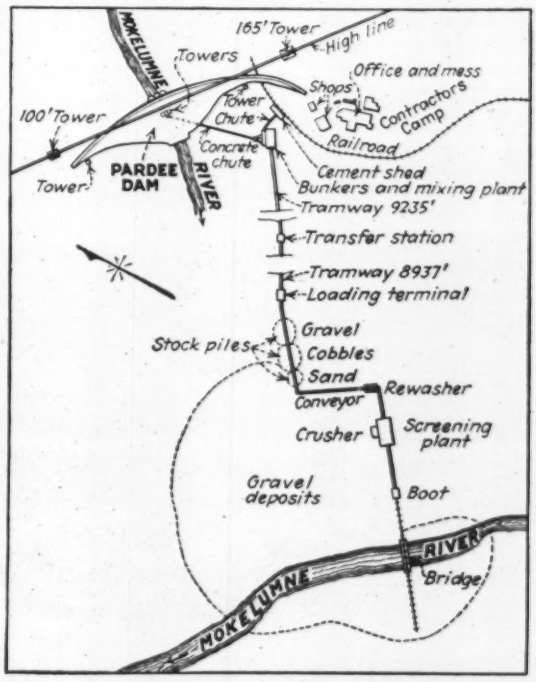

FIG. 3—MAP SHOWING GENERAL ARRANGEMENT OF DAM

The maximum capacity of the gravel plant is 2,000 cu.yd. of pit-run material per eight-hour shift; the average production is 1,500 cu.yd. per shift.

Cable Tramway—The problem of transporting the material for the aggregate from the gravel plant to the mixing plant at the dam site involved deciding whether to construct a 4-mile railroad with heavy excavation, a railroad 8 miles long with light excavation, or a tramway over a direct route of 18,700 ft. As the contractor is paid so much per cubic yard for the material placed in the dam, this was his problem. He selected the tramway now in operation.

A tramway with a lock-coil track cable is now in use.

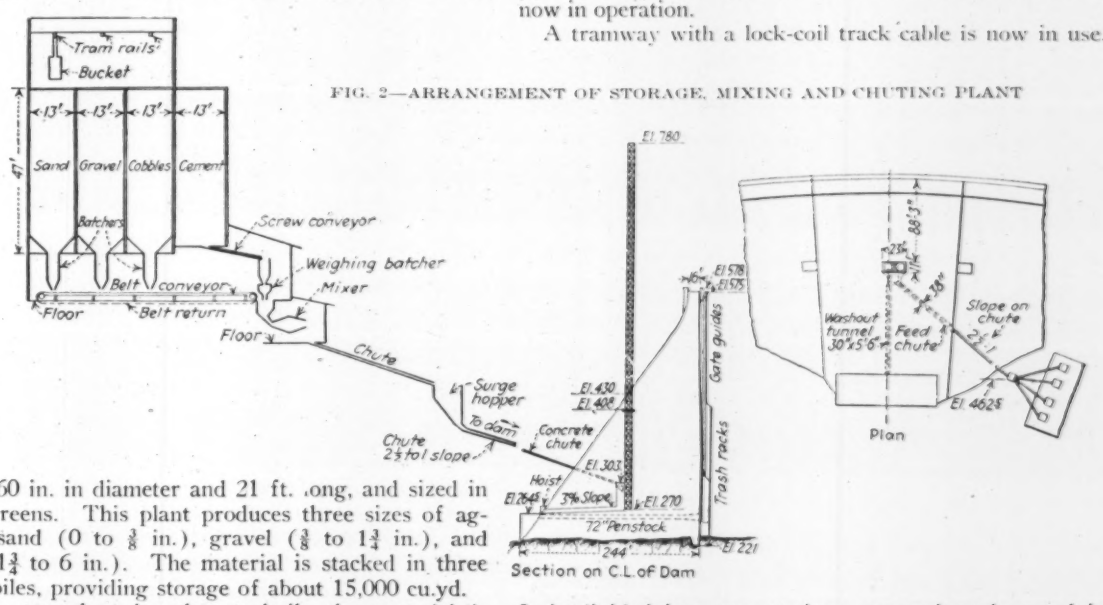

FIG. 2—ARRANGEMENT OF STORAGE, MIXING AND CHUTING PLANT

It is divided into two sections, approximately equal in length, with a transfer terminal between sections. There are 208 steel buckets on the line, each having a capacity of 30.2 cu.ft. The buckets are spaced at intervals of 192 ft., and one arrives at the terminal every 24 seconds. Each section is driven by a grip drive wheel, connected by chain belt to a 125-hp. induction motor. The horizontal distance from the loading terminal to the discharge terminal at the mixing-plant bunkers is 18,700 ft. The rise in elevation from loading terminal (El. 170) to discharge terminal (El. 574) is 404 ft., and the highest point on the line is 85 ft. higher than the discharge terminal.

The track cable varies from $1\frac{3}{4}$ to $1\frac{5}{8}$ in. in diameter on the loaded side, and from $1\frac{1}{8}$ in. to 1 in. in diameter on the empty side, having been designed for a working tension of 40,000 lb. per sq.in. of net section. The traction rope or cable is $\frac{7}{8}$-in. steel cable and travels at the rate of 470 f.p.m. The track cable is suspended with steel hangers from 40 wooden towers. The longest span is 1,960 ft., where the tramway line crosses from the north to the south bank of the Mokelumne River. On this span the $1\frac{3}{4}$-in. diameter track cable was required.

The tramway was designed for a capacity of 440,000 lb. of gravel, or 146.7 buckets, per hour.

Tramway operation began on Jan. 12, 1928, but, because of placing conditions at the dam, it was not until after Aug. 1 that full operating efficiency was obtained. During the 79-day period from Aug. 1 to Oct. 19 the tramway operated 235 shifts and transported 232,812 buckets of material from the washing plant to the bunkers at the dam site. On Oct. 19 it was shut down for fifteen shifts to permit the renewal of the hauling or tracton

cable. Up to that time a total of 459,822 buckets of material had been transported.

If the proper proportion of this time occupied in renewal of cable be applied to the percentage of total production obtained during the August-October period, we find that 232,812 buckets were transported in 242.5 shifts, or 960 buckets per eight-hour shift. Operation at 100 per cent efficiency, allowing fifteen minutes per shift for shutdown during meal time, would involve the delivery of 1,137 buckets per eight-hour shift. Because of shutdowns for repairs, renewals and other causes during the period, the actual operating efficiency obtained was 84.4 per cent.

As a basis for these figures the accompanying table showing the actual performance of the tramway is given.

SUMMARY OF TRAMWAY PERFORMANCE

Operations commenced Jan. 12, 1928

Period—1928	Total Buckets	Total Shifts	Maximum Buckets Per Shift	Maximum Buckets Per Day	Average Buckets Per Shift
Jan. 12–31	10,955	20.0	1,049	1,049	547.8
Feb. 1–24	30,698	40.7	1,137	2,295	754.3
Apr. 22–30	7,476	12.0	1,110	1,897	623.0
May 1–31	55,017	67.7	1,137	2,947	812.7
June 1–30	74,176	80.0	1,114	3,214	927.2
July 1–31	55,522	64.0	1,137	3,336	867.5
Aug. 1–31	91,640	93.0	1,135	3,329	985.4
Sept. 1–30	87,571	87.0	1,136	3,354	1,006.6
Oct. 1–19	53,601	55.0	1,132	3,357	974.6
Traction cable changed Oct. 19-24—fifteen shifts					
Total Jan. 12–Oct. 24	466,656	534.4	1,137	3,357	873.2

Figures from which the cost of operation can be estimated are available. The cost of electric power during August and September operations averaged $0.007 per yard of delivered material. The operating force of ten men had a payroll of $45 per shift, or $0.042 per yard, during the August-October period. Maintenance and repairs covering a period during which 198,000 cu.yd. was delivered averaged $0.096 per yard. This figure includes the cost of renewing the hauling cable at $0.016 per yard.

The cost items may be summarized as follows:

August-October (84.4 per cent efficiency)		Jan. 12-Oct. 19 (77.0 per cent efficiency)	
Labor	$0.042	Labor	$0.046
Power	0.007	Power	0.007
Maintenance and repairs	0.096	Maintenance and repairs	0.096
Total for selected period, yd.	$0.145	Total for operation to Oct. 19, 1928, yd	$0.149

The plant charge, retiring the capital investment, amounts to approximately $0.205 per yard. The total cost of transporting the aggregate material from washing plant to the bunkers at the mixing plant, a distance of 3.54 miles, is estimated at $0.354 per yard, or $0.26 per ton. This cost includes the cost of removal from the stockpiles, not the cost of stocking.

FIG. 5—CONCRETE-MIXING PLANT—MIXING FLOOR

One of the four 2-yd. mixers tilting. Three more mixers lie beyond. Each discharges into a chute, the four converging into a surge hopper which supplies the main chute.

Concrete-Mixing Plant—The washed aggregate in the three sizes mentioned, sand, gravel and cobbles, is delivered to bunkers at the mixing plant. These bunkers, built in one unit 80 ft. long by 40 ft. wide by 47 ft. deep to the top of the discharge tunnels, are subdivided into three deep and narrow bins, each 13 ft. wide. By the use of deep, narrow bins separation of the sizes within each bin is reduced to a minimum. The dumping on long sloping surfaces such as occur in shallow, wide bins is largely avoided. The total capacity of these bins is 5,200 yd. of sized material.

From the records it has been found that each bucket, containing 1.11 yd. of sized material, will furnish enough mixed aggregate to produce 0.714 yd. of concrete.

Four discharge tunnels containing the batchers used for measuring the aggregate run crosswise under the bins. Each tunnel taps the three bins. There are three aggremeters or batchers in each tunnel; one of 20- to 36-cu.ft. capacity under the sand bin, one of the same size under the gravel bin and one of 31- to 39½-cu.ft. capacity under the cobble bin. These batchers, measuring the material by volume, are successively discharged by hand to a 30-in. conveyor belt 65 ft. long from center

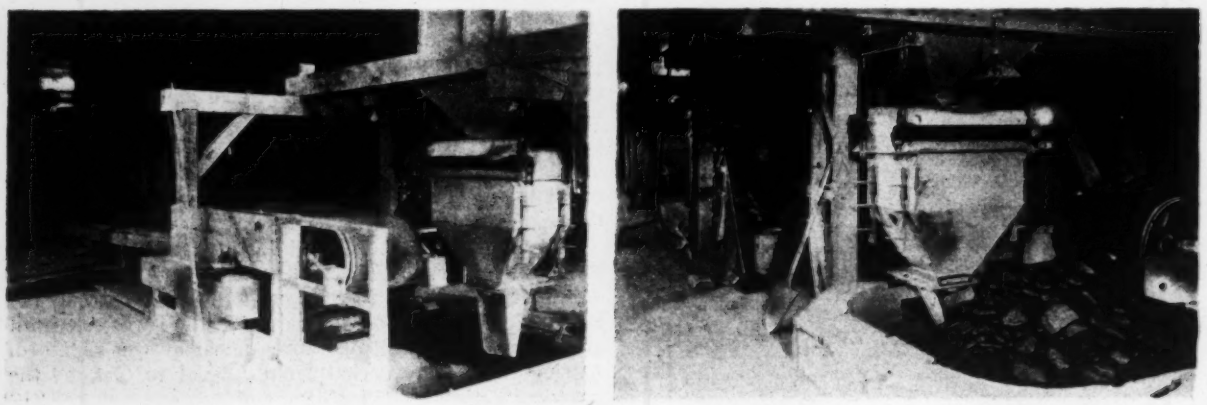

FIG. 4—CONCRETE-MIXING PLANT—CHARGING FLOOR

(*a*) Mixer hopper empty. At left, tunnel under bunkers, with gravel conveyor; at right, cement hopper over mixer hopper. (*b*) Mixer hopper full of typical charge with cobbles. View taken at right angles to preceding. There are four identical units of this kind.

to center of pulleys, which discharges the material into the hopper located above the concrete mixer.

Cement is brought in by rail to the mixer plant, a 5-mile branch line having been constructed by the contractor. It is received in cloth sacks and dumped from them by hand on an 18-in. conveyor belt which carries it to the head of the cement silo. This latter provides storage for 15,000 sacks of cement; it is rectangular in cross-section, 30x14 ft., and 40 ft. deep. Cement is stored in the silo only to a depth of 30 ft. By means of four 9-in. screw conveyors, the cement is conveyed to four twelve-sack weighing cement batchers, one located above each mixer hopper. The cement is weighed and discharged by hand into the hopper along with the three sizes of aggregate.

The concrete materials are mixed in four electrically driven tilting 2-yd. drum mixers. In operation they exceed their rated capacity, 2½ yd. of concrete being produced in each batch. Actual mixing time of 1½ minutes is required. The mixers tilt into discharge chutes which converge to an 8-yd. control hopper located on the hillside below the plant.

Equipment for Placing Concrete—The control hopper discharges through a manually controlled radial gate with an opening 16 in. square into a concrete chute 20 in. wide with egg-shaped bottom. This main chute slopes on a 40 per cent grade to the base of the main concrete tower.

The freshly mixed material flows through the main concrete chute to a hopper located near the bottom of a two-compartment main concrete tower. From there it is hoisted in 2-yd. tilting skip buckets operated by electric hoists to elevated hoppers attached to the tower, whence it passes by gravity through hanging counterbalanced troughs or chutes to tremie spouts locally known as "elephant trunks," through which it drops to the dam.

The main tower is a steel structure built on a concrete base within the dam section. It was originally erected to a height above its base of 330 ft., but recently it has been given new support using I-beams and channels placed in the dam itself at El. 408, which is 178 ft. above the old power base, and its height has been increased 180 ft., so that the tower now rises 510 ft. above its base and has its top at El. 780, or 205 ft. above the crest of the dam.

For use in placing the portion of the dam that will be above El. 500, the central tower has been supplemented by two single-compartment steel towers placed symmetrically on each side of the gorge with their bases at El. 520 and rising to El. 700.

A 10-ton "high-line" or cableway is an essential part of the placing equipment. It has a span on 1,280 ft. across the gorge and is equipped with a 2-in. track cable and ¾-in. hoisting line, operated by a 300-hp. induction motor.

Control on the concrete is maintained with cylinders taken daily from the main concrete chute at a point below the mixing plant and additional cylinders taken from the material already placed on the dam. All cylinders are taken by inspectors employed by the district. These cylinders are broken after seven days, 28 days, three months

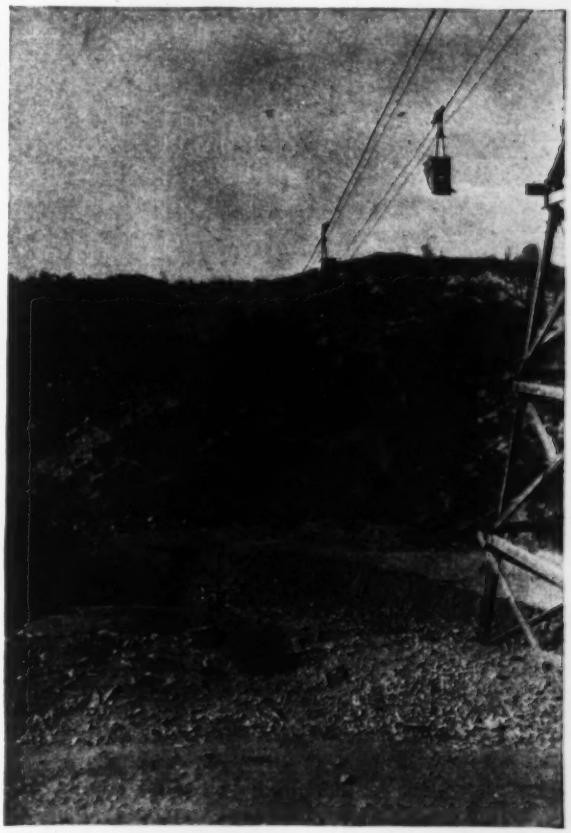

FIG. 6—CABLE TRAMWAY FROM GRAVEL PLANT
Span of 1,960 ft. across Mokelumne River. From here the tramway leads west 4 miles to the gravel plant. Bucket capacity 30 cu.ft. Tramway capacity 146 buckets per hour, handling 220 tons of gravel.

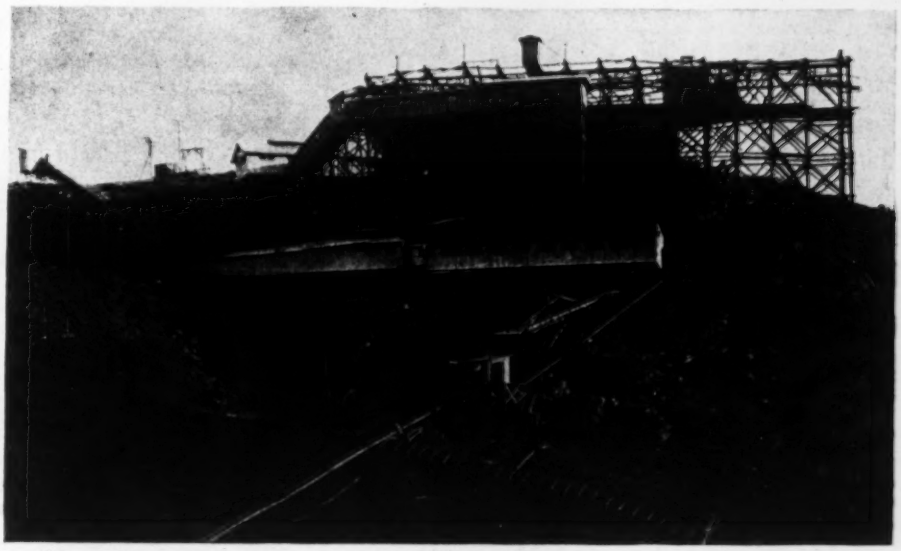

FIG. 7—PARDEE DAM MIXING PLANT
Sand and gravel are received by aerial tramway entering bunker shed at right; cement comes in by tunnel at left; four chutes from the four mixer units converge to surge hopper under small shed in center, when the concrete flows down chute leading to lower left corner. This chute extends through a hole in the dam to the main hoist tower.

FIG. 8—CONDITION OF WORK ON NOV. 7, 1928
Dam near half height. Concreting is in progress in the third section from the left.

and six months in a 200,000-lb. testing machine. The usual cylinders are 8x16 in., but at intervals cylinders 10 in. in diameter are taken for purposes of comparison. The cylinders taken from the material already placed on the dam are 6 in. in diameter.

Inundation of the sand is not practiced, but daily moisture determinations are made and the proper amount of water to be added is then calculated. To fix the total water, slump tests have been made, but much dependence is placed on the appearance and consistency of the concrete as it arrives at the dam. Water is measured into the mixers by using water meters equipped with dials having turn-back pointers, so that the water for each batch is directly indicated and at the same time a summation or total can be read from the dial. This permits the checking of the total quantity of water used against the total number of batches poured.

The dam design calls for concrete with 1,500 lb. per sq.in. compressive strength in 28 days. The tests on 8-in. cylinders taken during the past construction period indicate an actual strength obtained of 1,900 to 2,000 lb. per sq.in. in 28 days.

The concrete mixture contains 1 bbl. of cement per cubic yard. The typical proportions for a mixer batch are as follows:

Cement	846 lb.	(9¼ sacks)
Sand (0-¾ in.)	2,440 lb.	(25 cu.ft.)
Gravel (¾-1¾ in.)	2,113 lb.	(21 cu.ft.)
Cobbles (1¾-6 in.)	3,924 lb.	(40 cu.ft.)
Total batch	9,323 lb.	(95¼ cu.ft.)

As placed in the dam this will occupy approximately 63 cu.ft.

Methods Used in Placing—The concrete is placed in blocks between contraction joints located on radial lines. Below El. 400, or 167 ft. above the old streambed, these joints are 150 ft. apart. For the next 100 ft. of height, or between El. 400 and 500, the size of the blocks has been cut in half, so that contraction joints are 75 ft. apart. Above El. 500 the blocks are to be again divided in two, with the contraction joints but 37½ ft. apart. All contraction joints are recessed and are provided with vertical grout holes spaced 10 ft.

In placing the concrete, pours are made to depths of 5 ft. The forms, covered with sheet iron, are 5 ft. in height. On all pours a slope of 1 vertical to 20 horizontal from toe to heel is maintained by 1-ft. steps placed every 20 ft.

It has been the general practice to pour on the two night shifts each day, beginning at 4 p.m. and ending at 8 o'clock the next morning. Between 8 a.m. and 4 p.m. time is available for clean-up of the surface, placing forms and other necessary operations. Owing to delays due to the requirements of excavation and for other causes, this program has been varied and at times three-shift pouring with practically continuous placing has been resorted to.

By moving the "elephant trunk" from which the concrete spouts at frequent intervals, excessive flow of the concrete is avoided. A placing crew of fifteen men works over the concrete as it is poured, under the direction of an inspector. All placing is done in the presence of an inspector for the district.

On the shift following the placing of any layer of concrete, the surface is thoroughly brushed with stiff wire brooms to remove the laitance before it has hardened. A little later, before the next layer is placed, the old surface is given a thorough cleaning; picks and shovels are used, and everything is swept up with the wire brooms and washed away with a stiff jet of water mixed with compressed air. Before any concrete is placed on an old surface, cement grout is spread thinly over the surface, using wire brooms to work it into the irregularities of the surface.

It is expected that the placing of concrete in the dam can be completed about July 1, 1929. This will require a program for the future involving an average placement of 40,000 cu.yd. per month.

The general contractor is the Atkinson Contracting Company, of Los Angeles, Calif. The members of this firm are Lynn S. Atkinson, Guy Atkinson and William A. Kettlewell. The superintendent of dam construction is E. M. Whipple, and Henry Taber is engineer for the contractor. The design and the engineering supervision of construction have been handled by the forces of the East Bay Municipal Utility District, of which Arthur P. Davis is general manager and chief engineer, F. W. Hanna is chief designing engineer and James Munn is chief construction engineer. Located at the dam, the writer, is division engineer, with E. L. Macdonald as resident engineer.

Study Diatomaceous Silica as Admixture in Concrete

A study of the representative diatomaceous silicas commercially produced in the United States for use as admixtures in concrete is now being made at the Bureau of Standards for the purpose of developing federal specifications for this material. Eleven representative samples having been obtained from nine producers, the petrographic analysis, fineness through the No. 200 sieve and weight per cubic foot have been determined. The material runs from a minimum of approximately 70 per cent diatomaceous silica mixed with quartz and clay to practically pure diatomaceous silica. All samples are finely divided, from 86.2 to 99.5 per cent passing a No. 200 sieve. The determination of the weight per cubic foot introduced difficulties, the weight of any one material varying according to the method whereby it was placed in the measure. However, after careful experiments, it was found that the variation in weight per cubic foot was from 7.5 to 34.7 lb. Only one specimen weighed more than 16.2 lb. per cubic foot, six of the specimens varying from 14.3 to 16.2 lb. per cubic foot.

This Material—CONCRETE

An Article in Four Chapters

A Discussion of the Vital Problems of the Concrete Industry, Outlining Certain Obstacles to Consistent Quality Production and Indicating a Line of Progress

By Nathan C. Johnson
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III—Simplifying Quality Production

In the Two Preceding Chapters of this article a general view of the art was first had, with a pointing out of dissonances that commonly exist between the art as anticipated and the art as practiced. In the second chapter some reasons for these dissonances were pointed out; and in this present third chapter an endeavor will be made, by a study of a few fundamentals, to show how office and field and laboratory may be brought into closer accord and these dissonances avoided.

* * *

In any contractual relation there is assumed to be equity as well as a community of interest. For so much money a structure will be built as per plans, and subject, moreover, to the specifications as to quality, materials and workmanship.

Specifications, then, are peculiarly a part of the contract, in that their function is to assure the owner, through his representative, the engineer or architect, that he shall receive equity by getting a dollar's worth as evaluated by the specifications for his standard dollar. Any subversion of the specifications therefore tends to demonetize the owner's dollar; and a part of the engineer's function is to prevent this being done by substitutions or evasions.

Let the Engineer Do Equity

There is a legal maxim that "he who would receive equity must first do equity." This applies with peculiar force to the specifying versus the making of concrete. Making all allowance for the entry of human selfishness into the making of concrete as fully as into other affairs of life, it is none the less true that the obligation to "do equity" rests first of all on the engineer in the drafting of his specifications. If the engineer does equity in his requirements, the constructor may be called on to do equity in his performance, and the customary evasions of specifications and substitutions of just-as-goods will promptly be of no avail.

How May It Be Applied

It is the writer's observation and belief that both engineers and constructors desire most earnestly to have some way shown them by which they mutually may do equity and receive equity in matters pertaining to concrete. It is also the writer's observation and belief that some way or means beyond the common teachings is eagerly awaited whereby the seemingly unexplainable (and thus far unexplained) can be rendered comprehensible, so that costly accidents and misbehaviors may be eliminated.

It seems beyond question that this demand should be met, both in the interests of equity and also for the sake of progress in the art. And, as it may be conceded that thus far the mixing-up and molding and breaking of test cylinders has not yielded that necessary clue, the situation calls for the employment of auxiliary means, whereby an adjustment or reconciliation between theory and practice may be had partly by new knowledge, partly by greater clarity in the problem, and perhaps in greater part by the application of plain common sense to what at first appears to be an intricate problem.

Choosing the Ingredients

The first bit of the problem investigated will be the important one of selecting and prescribing and proportioning the sand and stone and cement used in concrete, for the composition of a mix affects both cost and the realization of specifications. And this must be investigated, for an inequitable specification covering these particulars may be unenforceable in fact no matter how much it may be in accord with theory; and an attempt to live up to it may break the constructor. This being so, we must also examine as to what justification an engineer has for uttering such a specification, unless he is prepared to have this clause and other clauses, whether good or bad, fall dishonored by lack of observance or by refusal of compliance.

The burden on the engineer to do equity in these matters is made greater because of the fact that, although specifications when drawn are flexible and may be corrected, the quality and value of any concrete are determined *within from four to six hours after the cement is wetted*. For better or for worse, within that interval, the compounding of the material, the mixing and the placing must be done; and no man living today can undo or make better what is then done.

The importance of proportioning the concrete as to its several ingredients should therefore be recognized by the proper wording of specifications. In drawing these specifications the engineer may say that strength at 28 days is the determinator of value and, in accordance with the accepted promulgations, compute or "predetermine" this strength by substituting particle-size analyses of sands and stone available on a given job in a formula derived from laboratory work; and thus, by working back through the water-cement ratio, determine the quantities of each material to use, including water, on the job, so that the concrete in the work both may and shall possess a certain and definite strength, usually at 28 days, and therefore be an embodiment of value.

As to Tolerances

As this procedure and its embodiment in specifications is now commonly assumed as one of the guarantees and

guarantors of permanent value in construction, as well as of the security for the design, it is not only worth while but also mandatory to examine what tolerances are both permissible and also necessary to be granted on the job in order that the constructor may carry out this predetermination in fact without the imposition of impossible hardships on the one hand or the sacrifice of the owner's equity on the other by the degradation of value that must follow a failure to achieve desired results.

It is indeed most essential that we seek to determine and define these tolerances, for one unfortunate thing about the promulgation of an absolute law or a formula without giving limits of tolerance is that, if the unmodified law and formula are right and equitable, any departure from them not only is a departure from equity but it is further precluded without a resultant failure to achieve the desired end. If the desired result is achieved regardless of the law or formula, then law and formula break down as an inevitable consequence, and all sense or feeling of equity is destroyed at the same time.

There is no need to indulge in story-book detective tactics to secure an admission at once from any thoughtful person that this law and formula for predetermination from gradings—and conversely, of gradings in sand and stone as an essential to quality—are violated right and left on commercial work, yet with the attainment of very satisfactory results, by and large. The harm done by this overthrow is not so much to commercial concrete as a commodity as to the standing both professionally and in equity of an engineer who words his specifications on these lines, and through his loss of standing to his profession.

Let us then seek either to justify or to correct this situation by an examination of the magnitude of the task imposed in the field by a specification that requires a definite grading because of this doctrine of the predetermination of strengths, which in turn rests on the assumption that this grading is necessary for the production of adequate results.

Simple Mathematics

To supplement the knowledge that we have from the testing laboratory with new knowledge, we can best employ mathematics, quite simple mathematics as contrasted with the intricate mathematics used in every-day design. Yet the more we use mathematics in the proportioning of concrete by a determination of the probabilities of realization and the chances of success the more strange it appears that mathematical computation, so essential to design, should have been either short circuited for some reason when we come to the making of concrete as a material or else never used.

In order to bring commerce and commercial usage into the picture, yet without excluding the laboratory, the unit quantity of cement best taken for this determination is the sack of 94 lb. net weight, or approximately 1 cu.ft. bulk. Also, the cement per cubic yard will be taken as six sacks; and the commercial unit of volume of concrete as the cubic yard.

In a bag of cement of 94 lb. there is 0.48 cu.ft. of solid cement, since its specific gravity is 3.10. When this 0.48 cu.ft. is ground up, fluffing and voids bring the volume to approximately 1 cu.ft., but the actual solid cement per sack remains this same 0.48 cu.ft.

By standard specification, the ground cement must be of such fineness that 92 per cent will pass through a 200-mesh sieve. That is, 92 per cent of this 0.48 cu.ft., or 92 per cent of 829.44 cu.in., is comminuted into particles that do not exceed 0.023 in. in diameter and thus have a maximum volume of 0.000000004 cu.in. each.

Counting in Billions

By standard specification, therefore, there must be 92 per cent of $829.44 \div 0.000000004$, or about 190 billion particles per bag as a minimum, or, in a six-bag batch of 1:2:4 concrete there must be 6×190 billion, or 1,140,000,000,000 particles of cement as a minimum. To make things simpler, we may, for the present, forget the remaining 8 per cent that fails to pass the 200-mesh sieve, with its hundreds of millions of somewhat coarser particles.

In recent endeavors to make cement more rapid in its reaction with water and thus get a high early strength, finer grinding than the specification calls for is followed. Some cements are of such fineness that 80 per cent will pass the 400-mesh screen. With such fine-ground cements the particles are, as a minimum number, just about ten times the above-mentioned numbers per bag and per cubic yard of 1 : 2 : 4 concrete, even forgetting for further simplicity the entire 20 per cent that fails to pass the 400-mesh, though much of this may pass the 200-mesh screen.

There may, therefore, easily be anywhere from 190 billions to one trillion, 900 billions of cement particles per bag; and anywhere from six times the lower figure to six times the larger figure as the approximate number of particles of cement per yard of concrete.

The mind fails to grasp these huge numbers, yet these are the numbers played with by laborers, yard after yard, on the job.

All this at first appears foolish to concern ourselves with, until it is remembered that by their combination with water these particles do the useful work in sticking sand and stone together to make concrete to resist stress to meet the needs of man in his various pursuits and purposes and so on, in a veritable House that Jack Built—to the value of many hundreds of millions of dollars each year.

If, then, in commercial work, $\frac{1}{2}$ lb. remains in the sack out of each sack, it is seen that we may be and quite possibly are short some 80,000,000,000 of these power units per yard; yet these tiny units, be it remembered, are the very life blood of concrete.

Varying Combinations

We begin here to get an inkling of why it may be that the laboratory, working with 50-lb. samples on test specimens under strictly regulated procedures, may, in substantial fact, predetermine strengths and derive formulas from repeated trials, while the field may fall down on it, if for no other reason than that the test specimens contain only one seventy-sixth as much cement as the yard.

In the laboratory, then, with no production time limit on preparing batches, the cement and other ingredients are easily handled and weighed and fussed with and may be kept comparatively constant, while on the job the cubic yard as a unit and hundreds of other yards on top of the first yard present a different problem. At this point, too, we may have found a reason why, even in the laboratory, individual specimens vary from one another by 8 per cent to 25 per cent, so that averages on from three to five specimens are required by standard specifications for a strength-test finding, rather than the out-and-out results on one specimen. What must we, then, expect in the field?

It is quite probable that a large percentage of those who read the foregoing are, at this point, bracing themselves for a tussle with some esoteric theory that has little, if anything, to do either with the equities or with the brutalities of everyday business. A mental tussle may possibly be necessary, but the dreaded theory and theorem will, in a few paragraphs, be found to be lacking as an academic quantity. On the contrary, we shall be quite speedily confronted with a most hard-headed and, if you will, a most brutal and every-day business proposition that is of direct concern to everyone concerned in the concrete art.

To vision more easily what is to come, a simple illustration will be helpful.

The illustration is auction bridge—52 cards of four suits of thirteen cards each and four players. Not many variables are involved. Yet the probabilities and chances of any player holding the same hand twice are not better than one in 635,000,000,000. It would seem poor judgment, with such odds, to invest much money or to demand that others put up much money on the chance of duplicating any bit of luck that chanced to come out of the pot on any occasion in bridge; yet in concrete millions and millions of dollars are put up, apparently at much greater odds and with good security. Why is it?

Add Sand and Stone

As compared with the low-numbered variables of the card game, the variables per yard of concrete transcend exact illustration and defy comprehension. Yet the cement particles per yard enumerated above are only one-tenth to one-seventh of either the volume or weight of other materials in concrete, and each of these materials—sand and stone and water—is not inert, as commonly supposed. Sand and stone, on the contrary, enter into the reactions that make up the salable article known as concrete quite as fully and quite as importantly, though not as voraciously, as do cement and water.

Their respective quantities are therefore important; and inasmuch as they are subdivided materials no less

VOLUME AND SURFACE AREA OF STONE AGGREGATE

Broken Stone

Screen Passed and Retained On	Number of Particles in 100 Lb.	Average Volume Per Particle, Cu.In.	Surface Area Per Particle, Sq.In.	Surface Area Per 100 Lb., Sq.Ft.
P 2 -R 1½	670	1.553	8.7047	40.5
P 1½-R 1	1,680	0.636	4.9261	57.47
P 1-R ½	5,680	0.1811	2.1139	83.38
P ½-R ¼	14,900	0.0692	1.1943	123.58
P ¼-R ½	43,820	0.0236	0.5339	162.47

Screened Gravel

Screen Passed and Retained On	Number of Particles in 100 Lb.	Average Volume Per Particle, Cu.In.	Average Diameter Per Particle, In.	Surface Area Per Particle, Sq.In.	Surface Area Per 100 Lb., Sq.Ft.
P2 -R 1½	430	2.4289	1.6678	8.7383	26.09
P1½-R 1	1,090	0.9509	1.2201	4.6764	35.39
P 1-R ½	3,830	0.2694	0.8013	2.0330	54.00
P ½-R ¼	10,030	0.1020	0.5797	1.0557	73.54
P ¼-R ½	37,330	0.0276	0.3753	0.4425	114.71

actually than is cement, these quantities mean numbers of particles; and to these numbers may be applied the rules and laws of mathematics, even as fully as in stress design.

It would be more or less a cluttering of these pages to put down in full these numbers of particles per cubic yard of concrete in any standard grading or any theoretical grading or any grading whatsoever, whether recommended by authority or not. But to aid those who may be earnest enough to do so, the accompanying tables are copied from a paper presented by L. N. Edwards before the American Society for Testing Materials in 1918, with grateful acknowledgement to Mr. Edwards.

To shorten matters, for after all we are not so vitally concerned with actual numbers in this discussion as we are with the picture as a whole, let us assume that, using the predetermination formula, the sand grains per yard are only 200 millions and the stone particles a mere 200,000 per yard as compared with, say, one trillion, 500 billion cement particles in a concrete of, say, 1 : 1½ : 3 volumetric proportions, predetermined as to sand and stone and designed for 3,000 lb. at 28 days.

The numerical set-up of this concrete per yard then is, by formula:

Cement particles, say	1,500,000,000,000
Sand particles, say	200,000,000
Stone particles, say	200,000

What chance is there, then, under the application of the mathematics of probability and chance, that with these assumed and rather understated numerical variables alone considered, one yard may be like the next, no matter how well they may be shuffled?

VOLUME AND SURFACE AREA OF SAND

Sieve Passed and Retained On	Number of Particles Per Gram	Average Volume Per Particle, Cu.In.	Average Diameter Per Particle, In.	Surface Area Per Particle, Sq.In.	Surface Area Per Gram, Sq.In.
P 4 -R 8	14	0.00166	0.14695	0.0678	0.9492
P 8 -R 10	55	0.0004226	0.0931058	0.0272335	1.4978
P 10-R 20	350	0.00006645	0.0502534	0.0079338	2.7778
P 20-R 30	1,500	0.00001549	0.030928	0.0030055	4.5083
P 30-R 40	4,800	0.000004842	0.0209893	0.0013841	6.6432
P 40-R 50	16,000	0.000001452	0.0140818	0.0006201	9.9216
P 50-R 80	40,000	0.0000005815	0.0103568	0.000336913	13.4765
P 80-R 100	99,000	0.0000002348	0.0076538	0.000188325	18.6442
Standard Ottawa	1,323	0.00001757	0.0322545	0.00326838	4.3241

Now Add Water

The foregoing is the set-up in the dry; but let us say that a further part of the standard design of this mix calls for the use of 7+ gal., or almost 1 cu.ft., of water per bag of cement, in order to carry out the water-cement ratio manipulation of the formula. The exact quantity is purposely assumed as of minor importance in this illustration.

We may convert any given volume of water into particles readily enough. Every steam exhaust does it. Nature does it in clouds and fog; and for purposes of illustration and computation, we also may do it, figuratively at least.

Since the cement of a bag is 0.48 cu.ft., the water-cement ratio by volume is found at once to be 2.08 : 1. The particle ratio of water to cement is therefore numerically the same if equally subdivided, so that the set-up of the yard of concrete, with all materials in the mixer drum, would be as follows:

Water particles, say	3,120,000,000,000
Cement particles, say	1,500,000,000,000
Sand particles, say	200,000,000
Stone particles, say	200,000

There is no purpose, save as a matter of deep interest, in applying the formulas of probability and chance to these colossal numbers. The result is an incomprehensible series that must be expressed in logarithms to find space on a line clear across the page. Yet it is recommended that those who desire a full understanding of concrete carry out such a computation, for it is a most useful and illuminating application of mathematics. The actuarian tables of insurance companies, for instance, are based upon it, and insurance is unquestionably a profitable enterprise. The recent perfection of the radio tube has taken account of this same use of mathematics; and the possible rewards this year to the tube

makers are in excess of $20,000,000—thanks in part to these formulas. Concrete and concrete men might well sit up and take notice.

An office computation of design of mixture has therefore very, very much less than one chance in a million of being applicable to or realizable in commercial quantities and in field procedures, even dividing the probabilities by another million. This is a fair and moderate statement.

Field Factors

Why, then, do we succeed as well as we do in the field and why do we so often fall down in the field under current standards in specifications?

Part, at least, of the answer lies in the figures above tabulated for the numerical make-up of a yard of concrete.

First of all we have, possibly for the first time, a real visualizing of our old friend "excess water." A cement particle (or a whole bag of cement) does not react best with more than twice its volume of water; and any excess over the quantity required for best reaction is, technically, excess water. The usual assumption used in the water-cement ratio is volume of water (in gallons) to volume of *fluffed* cement, including voids, which gives a distorted ratio and picture.

Second, by mating cement and water particle for particle, we see that any increase beyond the minimum quantity commercially required speedily becomes a super-excess of water; and we see further that this is particularly true if a careless laborer or a vagrant breeze deprives any mix of a few hundred million particles of cement. But of the two materials, cement and water, we also see that both by numbers and by nature the quantity of cement per batch is a more critical factor than is the water, by reason of the particle numbers of each present in a mix, if for no other.

Certain Effects of Water

In subdividing water into fog, we have been taking liberties with that substance which Nature will not, in actuality, permit, nor will commercial procedures. We cannot produce plasticity or workability in concrete with fog, nor will water continue to exist in such subdivision. On the contrary, the fog droplets coalesce at the slightest excuse and form that fluid of peculiar and remarkable properties that, at ordinary temperatures, is known as water.

One of the most valuable properties of this fluid in concrete is lubrication of the mass. If it were not for this lubrication we could not place or mix concrete with any degree of success, let alone economy.

And a second property of water—a property that is the savior of the whole situation in the making of concrete and one possessed by water in high degree—is that of diffusing, or spreading evenly throughout itself, substances which dissolve in water. If it were not for this property, each cement particle in a mix would have its radius of action limited to about a millionth part of an inch and would be both useless for and incapable of sticking sand and stone together to form concrete.

So it is seen, as a third point, that we must have enough water in a mix, both for lubrication and for diffusion; and both requirements alike are severely practical. Enough water is, in fact, preferable to too little water; and cement, equally with constructors, will not submit to the water-starvation of a mix. But cement will equally not submit to an excessive quantity of water, for diffusion stops quickly when there is too much water.

The "Mother Liquor"

Although mention of these foregoing factors and facts is, in a sense, out of place in this chapter, and although more extensive mention of them must be made in the next and concluding chapter, it should be understood at this point that the setting and hardening processes of portland cement and the building-together of the numberless particles of sand and stone in any given mass are due, first of all, to the quick formation of a sort of "mother liquor" by the first and almost immediate reaction of cement and water; and second, to a profuse production of secondary products of a certain character such that all these hydration products, some singly and others together, diffuse or spread through this mother liquor in a manner that gives us a pretty good final result regardless of the apparent chances against this event happening.

We should be and are thankful that this is so. But we should at the same time recognize that this good fortune is neither of our making nor caused by any acts or virtues of ours, so that we should be correspondingly tolerant in our assertions and in our specifications. But since this is so, and since it is further so improbable that office computations of strength can be realized, because of their utter dependence on the conformity of field facts to office assumptions, it would seem better and more worthy of engineering and engineers to seek out a way less arbitrary and at the same time more certain and more equitable to all than is the present accepted procedure as embodied in specifications *which cannot be enforced in fact*.

Whatever may be individual opinions in this regard, there surely can be no difference of opinion, or quarrel, or blinding of the eyes, or separation of minds, or lack of equity, if in this important matter of proportioning a concrete mix there is found a way to secure actual and practical concurrence between the laboratory and the specifications and the actual work of quantity production by the constructor in the field.

This, if attained in fact, would be an important step nearer standardization and a practical realization of those economies that are so highly to be desired and so necessary in this competitive era.

Practical Relations

To this end and looking again at the particle composition of a yard of concrete, our minds at once revert to the need for accurate measurement. Accurate measurement of all materials is quite obviously a general necessity and a particular necessity as regards water, cement and sand. The whole affair can be easily upset if there is inaccuracy in measurement. As to the quantities of each of these materials best employed, we may investigate in the same order and determine rather easily a fair, yet inclusive, specification.

Water, as may be seen from the tabulation, *is always in excess in a mix*, so far as chemical reactions and diffusions are concerned. As witness of this, we have the fact that whereas cement and water alone, in paste form, will attain a strength at 28 days of from 12,000 to 16,000 lb., yet the mixture of cement and sand and of cement and sand and stone is far below this in strength, although the stone and the sand individually are worth from 30,000 lb. to 60,000 lb. per sq.in. in compression. The reason

for this failure to realize in the mixture the average of its components is, of course, the slowness of diffusion of cementing products in the excess of water required for commercial plasticity.

We may as well bow to this unless we devise and use auxiliary substances. Further, since concrete out of forms is not only useless but an impossible nuisance, we must compromise with science and permit, in our specifications, *enough water to give a plastic or mushy consistency, regardless of its absolute quantity.*

It is well to remember, as pointed out in the first chapter, that there is always sufficient strength in concrete to sustain its designed load (and *many times* its designed load), unless we have a revision of the code that will permit the use of smaller sections with less quantities of material. But it may be confidently stated that there is no likelihood of that revision for many, many years to come.

As to Sand

But we may do something as to water by proportioning the other materials of concrete. Sand and stone are the materials that make concrete difficult to place; and sand and stone are the materials that demand lubrication by water. So let us examine them as to their individual and combined properties.

Reverting for this purpose to our numerical set-up of sand and stone particles per yard and again referring to the Edwards table, it is seen that just as cement is quantitatively more sensitive than water, sand is quantitatively more sensitive than stone. That is, sand: cement is a far more critical ratio in proportioning than is cement: stone or cement: water.

As to proportioning, it will also be observed from the table of Mr. Edwards that the larger any particles are the fewer there are in a given volume or in a given weight. That is, the larger the particles the less the number of variables we shall have per yard. This simple observation at once connects up with some of the older principles of the art, such as the obsolete specification requirement that "sand shall be sharp and coarse." Coarse sand *is* sharp sand, inasmuch as it is the absence of fines that makes a sand seem sharp to the touch; and the absence of fines means fewer variables and makes more uniform concrete. Furthermore, there is an old adage that "crushed stone makes better concrete than gravel." This is not necessarily true, but it undoubtedly had its foundation in the observed fact that, before the doctrinism of grading permitted the crusher dust and all else to be included in deliveries, crushed stone was of substantially one size, thereby removing a few millions or billions of variables from the composition of any yard of concrete that was made with crushed stone, whereas unscreened gravel, in several sizes, carried in these variables and thus made poorer concrete.

How Much Stone

This may be rephrased, then, to establish a fourth principle, by saying that stone particles, unless too large to make a workable mix, can be increased almost without limit, up to the limits of good common sense.

In determining what is this good common sense, the laboratory is most helpful. Absolutism, and control of quantity and of manipulation, are necessary for this determination; but in view of the vast number of tests that have been made and recorded, and not least the averages from these tests, we may readily refer to published data and determine that under the conditions occurring in those tests certain things are evidently good common sense and certain other things are not.

The puzzle of concrete and its making fits together rapidly if all these apparently unrelated and contradictory factors are now considered in this new light and without the inhibitions and fearsomenesses inseparable from any attempt at following an impractical course.

Some Conclusions

First, then, we see from the counting of cement particles and of sand particles and stone particles that the abstraction of even a small amount of cement robs us of a vast number of power units which we may need very vitally in our finished product. In plain terms, the lesson is to use enough cement and not to seek to save cement beyond a certain safe minimum, regardless of test strengths obtained below that minimum, as perhaps by trick gradings from laboratory formulas, with final reference to field test specimens made in uncommercial ways, to prove that a deficiency of cement is justified.

Second, we see from the evidence of hundreds of thousands of tests that the relative amounts of sand and cement should not exceed two parts of sand to one of cement, for the variables increase vastly by the inclusion of more particles of sand, and the chances of success diminish as these increase. Sand above the 2:1 ratio may be termed "the devil in the cheese" of concrete, both as regards strength and as regards permanence.

Third, since stone (or gravel) particles are of relatively large size as compared with sand particles, we see again that a large increase in their volume or weight proportion has but little effect on the variables of the mix. Further, stone, if of good quality, is the better of the two best materials in concrete, sand being a poor second best. Therefore use all the stone possible; and since ease of placement comes in as an economic and commercial factor, the amount of stone used and the size of stone must be such as will admit of ready placement in forms.

An Outline Specification

Laboratory and field, then, can come together on this common ground, and they both can agree on the equity of a specification that would read somewhat as follows:

1. Use one part of cement (as a basic requirement).
2. Use not over two parts of sand if the sand is coarse and less if the sand is fine (so that a common sense ratio shall not be exceeded).
3. Use as much stone as the mix will comfortably carry; and
4. Use only enough water to give mushy or workable consistency and without separation. (This will take care of the varying water content of the aggregates, yet permit of the necessary lubrication of the mix.)

Then, from the lessons of the test cylinder and its specifications as derived in Chapter II, there would come the following:

5. Place the concrete, after thorough mixing, evenly in forms and puddle into place in even layers throughout any given section; and (from the same source)
6. Fill each section full to the top in a continuous pour (so as to bring the field concrete into some resemblance of the test specimen).

The foregoing specification applies well to either mass or reinforced-concrete work. It upsets nothing, it opposes nothing. It is non-controversial both in derivation and in its application to practice.

This specification recognizes the value of strength and goes as far toward insuring strength in the field as is possible. This specification also recognizes both uniformity and workability and leaves the road open to insure both. It also recognizes economics and the value of the water-cement ratio, but it further recognizes that materials differ at different localities and in different seasons, and oftentimes from delivery to delivery. It has proved to be a very good specification in practical work.

The derivation of this specification in the first instance was from the observation of the processes of Nature that takes place in the mass. Stones sink. In any concrete this takes place and it is Nature's endeavor to bring about as great a compacting of the mass as is possible. For this reason the apparent excess of stone called for by this specification over 1 : 2 : 4 or like proportions has proved successful.

The full and final derivation of the specification is due to the co-operation of many earnest field men and many earnest engineers and architects who have helped in its working out. The writer has no personal interest in its general adoption beyond the realization that although few of us can afford to bother about the excellence or lack of excellence of the other fellow's concrete, none the less every job that results unfortunately is a blow to the entire field of those who work with concrete, for what affects one affects all.

And not least, under such a specification the contractor knows at the time of bidding how he may economically produce what is intended; and there is neither need nor demand for unstatable or unstated tolerances, or changes after the event. Under the provisions of this specification the contractor may use, with the justifiable sanction of the engineer or architect, the materials available in a given locality, yet with the attainment of as perfect a result as the materials themselves admit. And not least, while attaining a good and practical result, the constructor can never be crucified under such a specification, because it is definite.

Getting into accord with Nature in the matter of proportioning and making concrete is of no small importance. Accord with Nature may be defined as science; and discord with Nature can only bring misfortune, for, compared with natural laws, the laws and opinions of mankind are puny and futile.

In the next chapter of this article a further endeavor will be made to seek out the ways of Nature as related to the formation of the cementing material that is produced by the union of the dry and powdered cement with water. A few studies will be made as to the nature of this hydrated material, what we may expect from it and how we may advantageously use it and to better effect than we commonly are doing at the present time.

The fourth and concluding chapter of Mr. Johnson's article will appear in the issue of Feb. 21.

Tolls Across Texas Bridges Reduced

Acting upon complaints filed by the highway commissions of Oklahoma and Texas that excessive tolls are charged for passage over the bridge owned by the Gainesville Red River Bridge Company of Texas across the Red River near Gainesville, Tex., and for passage over the bridge owned by the same company across the Red River near Denison, Tex., the Secretary of War, after an investigation, has found the complaints justified and has directed a reduction of the tolls under authority of an act of Congress approved March 4, 1927.

Designers Differ With Board on Bridge Failure Report

Fix Abnormal Friction From Inferior Quality Bronze Bushings and Faulty Lubrication as Prime Cause of Hackensack Bridge Failure

AN INDEPENDENT investigation made by the Strauss Engineering Corporation, of Chicago, on the wrecked east leaf of the double-leaf bascule bridge built from its plans and specifications for the New Jersey State Highway Commission over the Hackensack River near Jersey City, N. J., leads it to conclusions regarding the failure that are different from those found by the special investigating board and reported in *Engineering News-Record*, Jan. 10, 1929, p. 74. (A news account of the failure which occurred while the bridge was being closed on the night of Feb. 15 also was published in *Engineering News-Record*, Dec. 20, 1928, p. 931.)

As a result of its examination the special investigating board concluded that "Failure was due to inadequacy of strength of members in the counterweight supporting and bracing system. Initial failure occurred in the north leg of the counterweight tower, and fracture of this tower leg had been progressive for some time. It is impossible to tell if the counterweight columns failed. It seems certain that imperfect lubrication of the tail bearing, that pier sinking and that any unusual method of bridge operation at the time of failure did not contribute to the cause of the accident." The board also reported that test operation of the undamaged west leaf showed material deflections and abnormally high stresses in the counterweight towers. On the recommendation of the board these towers were reinforced before the bridge was again placed in use.

The Strauss company, as a result of its investigation, which included an inspection of the wrecked structure, a metallurgical analysis of the bronze bushings of the tail bearings and a review of its correspondence relating to the erection of the bridge, concludes that "There can be no doubt that the prime cause of the failure was the abnormally high friction developed at the counterweight trunnion bearing (tail bearing) of the north truss; that this high friction was due to operation while the trunnion was not properly lubricated and that the condition was aggravated by the poor quality of the bronze bushings." A complete report of this latter investigation has been sent to *Engineering News-Record* by C. E. Paine, vice-president and managing engineer of the Strauss Engineering Corporation, the essential parts of which follow.

Some important facts bearing on the failure of the east leaf of the bascule span over the Hackensack River on the Lincoln Highway between Jersey City and Kearny, N. J., are not mentioned in the preliminary report of the board, and they lead us to conclusions differing from those indicated by the board. The Strauss Engineering Corporation furnished plans and specifications to the New Jersey State Highway Commission for the superstructure of the double-leaf vertical overhead-counterweight bascule. Inspection of materials was under the jurisdiction of the state, which also supervised construction.

Following the accident a representative of our company made an inspection of those parts of the structure which remained on the pier. Examination of the main trunnion and main trunnion bearings on the south side of the leaf disclosed wooden plugs in the ends of the lubrication grooves in such a position as to shut off completely the principal source of lubrication. The sliding surfaces of this main

trunnion showed evidences of having been operated at some time in the past without lubricant. Incidentally during the operation of the west leaf on Dec. 18 when tests were being made by the investigating board the counterweight trunnions were turning intermittently in their bearings and apparently were developing very high friction.

Subsequent investigation after portions of the wrecked structure had been removed from the river disclosed that the bushings of the main trunnion bearings, the main trunnions, the bushings of the counterweight trunnion bearings and the counterweight trunnions all showed large patches of a red copper-like film distributed over the sliding surfaces. On the main trunnions the lubrication grooves were plugged, and tool marks were found on what was supposed to be a polished surface of the bushings of the counterweight trunnion bearings.

The condition of the trunnions and bearings taken from the east leaf as well as the intermittent jerky movement of the counterweight trunnions of the west leaf indicate that conditions were far from normal as regards development of friction. The presence of the wooden plugs in the lubrication grooves of the main trunnions points toward a general condition of unskillful workmanship in the assembling of the structure. The copper-like film on the bearing surfaces of the trunnions and the burned sliding surfaces of the bronze bushings show conclusively that at some time in the past the parts were operated with little or no lubrication and that sufficient heat was generated by friction to braze the bearing metal onto the trunnions. This is indisputable evidence that at some time the counterweight trunnions of the east leaf developed friction far in excess of that normally encountered and at that time the fracture (in the north tower) was started.

Inferior Bushings

Another fact contributing to the high friction realized on these bearings is the inferior quality of the bronze bushings. Samples were taken from the bushings of the counterweight trunnion bearings by Dr. R. S. Williams, professor of analytic chemistry and metallography at Massachusetts Institute of Technology, who has investigated this metal.

(Following are the conclusions and principal data in Dr. Williams' report to the Strauss Engineering Corporation.—Editor.)

"*Chemical Analysis*—Chemical analyses taken from the different sections of the counterweight trunnion bearing show marked variations in the composition of the alloy, indicating lack of uniformity in the cast metal. The analysis of the metal is approximately as follows:

	Per Cent		Per Cent
Tin	About 15	Copper	82.7
Antimony	About 1.7	Iron	Small amount

"The presence of antimony is not to be expected in a bearing metal of this class and was not indicated in the specifications. No information can be found as to the effect of antimony, but it is my opinion that it would be distinctly harmful. The accompanying photomicrographs indicate the unusual character of the metal.

"*Conclusions*—The following facts are indicated:

"1. The bearing had operated at times either wholly dry or quite inadequately lubricated.

"2. Metal-to-metal contact must have occurred between the steel trunnion and the bronze bearing.

"3. The chemical composition of the bearing differs from the usual bronze bridge trunnion bearing in its percentage of antimony, and it is my opinion that the anti-friction properties of this alloy are less than could be expected from the normal bronze.

"4. The composition of the alloy varied in different parts of the bearing, indicating lack of proper mixing in the foundry.

"5. The porosity of the casting shown by the number of large holes indicates bad casting practice.

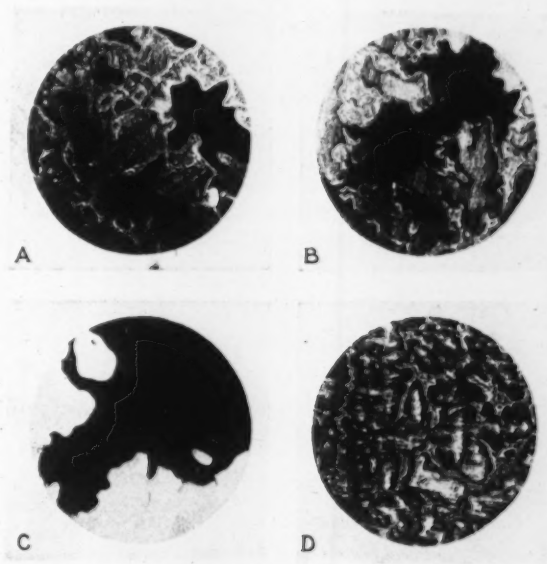
"6. The microstructure of the bearing metal is not usual, and in my opinion the alloy is not as well adapted for bearing purposes as a metal showing a more normal structure.

"7. It is my opinion that at times the friction on the counterweight trunnion bearing was greatly in excess of that assumed by the designers."

A review of our files reveals that trouble was experienced in the manufacture of the bronze, that we recommended rejection of trunnion bushings on several reports and that the State Highway Commission did order them rejected. In spite of this the records further show that four trunnion bearings were shipped without inspection and that the inspectors found these bearings in place on the trunnions at the bridge site and were able only to make a surface inspection. We suspect that these bushings were part of those which were rejected.

Conclusion—No combination of the forces and resistances normally encountered in the operation of a bascule bridge or laid down in any standard specifications could cause unit stresses which will account for the failure. Whether or not there were any inaccuracies in the construction of the counterweight or in the assembling of the elements of the counterbalancing system which might have contributed to the failure it is now impossible to determine. But in view of all of the facts and evidence reported there can be no doubt that the prime cause of the failure was the abnormally high friction developed at the counterweight trunnion bearings of the north truss; that this high friction was due to operation while the trunnion was not properly lubricated and that the condition was aggravated by the poor quality of the bronze bushing.

If a highway bridge designed for automobile traffic were subjected to the loads incurred by the attempted passage of a railway train, it would undoubtedly fail and the statement might be made (and correctly) that it failed because it was structurally weak. A more accurate and certainly a more illuminating statement would be to say that the bridge failed because it was subjected to loads far in excess of those for which it was designed or for which it was ever built to carry.



PHOTOMICROGRAPHS OF BRONZE IN BEARING BUSHINGS

A shows large dark brown dendritic masses (black in the photograph) with which are associated fine-grained eutectoid and glistening white fibers believed to be due to antimony. In *B* the large black holes show the unsound character of the castings. *C* is an unetched section showing a large irregular hole in the casting. *D* may be compared with the other photomicrographs, since it shows the structure of a copper-tin bronze free from antimony. The cooling rate is probably not the same as in the bridge casting, hence a comparison on the basis of grain size is not justified.

Tax-Exempt Bonds Increase in Number

Wholly tax-exempt bonds outstanding on Nov. 30 1928, according to estimates of the U. S. Treasury Department, amounted to $17,115,000,000, representing a gain of $108,000,000 over Oct. 31. States and their subdivisions accounted for $87,000,000 of this increase.

Geophysical Methods for Subsoil Studies in Foundation Work

Two Articles Outlining the Adaptation of Modern Scientific Procedures to Important Requirements of Civil Engineering Construction

EXPLORING underground conditions by other methods than test boring has come to have a recognized field in mining work. Only recently, however, has it been recognized that quite similar data are required in the work of the civil engineer, as in studying foundation conditions for dams and other structures.

Two interesting methods available for such investigations are described in the following articles: an electrical resistance method and a seismograph method. The former has already found useful practical application in at least two cases; the seismograph method is still restricted to mining (oil-well) work, but is claimed to have extensive possibility for foundation study.

* * *

Electrical Subsoil Exploration and the Civil Engineer

Foundation Conditions at a Dam Site Investigated by an Interesting Method Which Checked Closely With Results of Borings

By Irving B. Crosby
Consulting Geologist, Boston, Mass.
and
Sherwin F. Kelly
Schlumberger Electrical Prospecting Methods, New York City

ENGINEERS must often wish they could see into the ground and learn what the conditions are beneath their feet without waiting for the slow and expensive processes of drilling and digging. The geologist has often been of aid on such occasions, but now his efficiency is greatly increased by the use of a new tool, geophysical exploration, which the physicist offers to the geologist and the engineer. One very important application of geophysical exploration to practical problems is in determining the depth to bedrock without drilling or digging.

The introduction of geophysical methods of geological investigation into civil engineering has come about only very recently, although they have been utilized in mining exploration for more than a decade and a half. Among the various methods of proved value, the electrical method possesses certain advantages of simplicity, mobility, flexibility and economy.

The potential method of electrical exploration was developed by Conrad Schlumberger, mining engineer and formerly professor of physics at the School of Mines in Paris. For those interested in other phases of the potential method the following references may be cited: (1) C. Schlumberger, Etude sur la prospection électrique du sous-sol; Gauthier Villars, Paris, 1920. (2) C. and M. Schlumberger, Phénomènes électriques produits par les gisements métalliques; *Comptes rendus* de l'Académie des sciences, Vol. 170, p.519, Paris, 1920. (3) E. G. Leonardon and Sherwin F. Kelly, Exploration for Ore by Potential Methods; *Transactions* of the Canadian Institute of Mining and Metallurgy, Vol. 30, Montreal, 1927. (4) E. G. Leonardon and Sherwin F. Kelly, Some Applications of Potential Methods to Structural Studies; American Institute of Mining and Metallurgical Engineers, Technical Publication No. 115, New York, 1928. (5) G. Carrette and Sherwin F. Kelly, Discovery of Salt Domes in Alsace by Electrical Exploration; *Mining and Metallurgy*, New York, September, 1928.

Practical application of the method to certain foundation problems demonstrates how effectively it may be used for the investigation of bedrock concealed by overburden.

The Method—The basis of the method is the fact that the various materials constituting the earth's crust offer different degrees of resistance to the passage of an electric current. If a current flows through the ground it will create therein an electrical field, which will be a regular one when the ground is homogeneous. On the other hand, if there is a mass present which is more resistant or more conductive than the adjacent material, then this electrical field will be distorted. Bedrock is frequently more resistant than the overburden, and due to this fact can be differentiated electrically.

This is done by means of a direct electric current

ELECTRICAL EXPLORATION IN THE CANADIAN WINTER

passed between two earth contacts at the surface of the ground. The potential at any given point can be mathematically calculated if the earth is perfectly homogeneous. Near each earth contact the equipotential surfaces about it are practically concentric and hemispherical. With increasing distance from this contact these concentric "bowls" become progressively distorted and shifted in a regular manner.

Fig. 1 gives an ideal sketch of the traces of some of these bowls at the surface of the ground. As already noted, in the case of a homogeneous and isotropic medium, the form and potential of each curve are known. Suppose, now, a mass of indefinite extent (No. 2 in Fig. 2) lies beneath a uniformly thick layer of higher conductivity (No. 1 in Fig. 2). It is evident that the equipotential surfaces near the contact *A* will not be hemispheres, as in the case described above, but will have quite different and more complicated shapes. At the boundary between the two layers refractions will take place in these equipotential surfaces, and the bowls will be distorted in an entirely different manner from that mentioned above.

These distortions will be reflected in the shapes and spacing of the curves at the surface of the ground, with the result that the potential drops on the ground will also be different from those observed in homogeneous material. This fact provides the key to the problem of determining the thickness of the first layer. It is

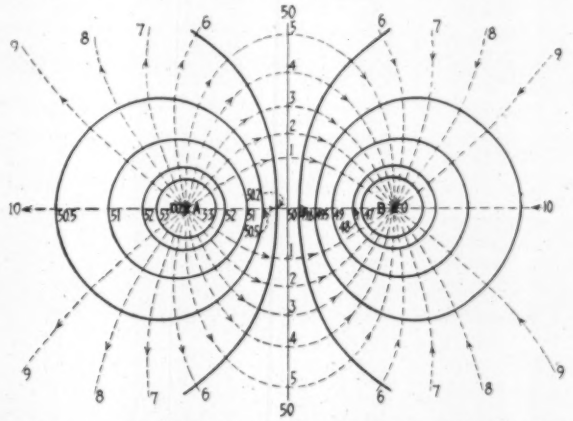


FIG. 1—EQIPOTENTIAL CURVES AT SURFACE OF HOMOGENEOUS GROUND (SOLID LINES)

possible to demonstrate satisfactorily that this problem is one capable of exact solution.

The field technique for gathering the necessary data consists of passing a current through the ground between the two earth contacts, as described above, and then with movable electrodes measuring the drops of potential between successive points along one or two lines radiating from the position to be studied. The simple principles underlying "electrical drilling" are affected by complications in case of a larger number of strata of varying characteristics, and in such case only an approximate solution can be reached; but whenever the beds are horizontally homogeneous and well differentiated vertically, the problem can be solved.

The factors affecting the disturbances observed in the field of potentials are distance of disturbing medium from the surface, thickness of the various beds, and relative conductivities of these beds. A very thin layer cannot, of course, be detected at great depth.

The representation of the results is generally made by translating the field data into the resistivity values of the formations at successive depths. These resistivities are then plotted as ordinates against the corresponding depths as abscissas.

A Dam Foundation—This technique was applied last spring, upon the initiative of the senior author, to the study of two dam sites on the Connecticut River near Littleton, N. H. This is the Fifteen Mile Falls development of the New England Power Association. The valley is underlain by a deeply buried preglacial gorge filled with glacial deposits. In order to speed up the

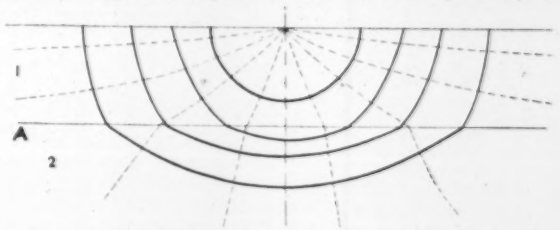


FIG. 2—DISTORTED EQUIPOTENTIAL CURVES (SOLID LINES) AND CURRENT PATHS (DOTTED LINES) A horizontal bed overlying a mass of higher resistivity.

work and cut down the cost of studying the location of the buried bedrock, geophysical methods were called in to supplement the drilling, which proved slow and expensive because of the boulders in the overburden.

The glacial drift in the buried gorge consists of till or boulder clay, gravel, sand and clay. The bedrock is largely chlorite schist, a metamorphosed rhyolite, hard and dense, probably pre-Cambrian in age.

The problem consisted of determining the depth of overburden at various points in order to prepare a contour map of the bedrock surface and thus outline the buried gorge at the sites chosen. The two locations are 8 miles apart and presented somewhat different subjects for investigation. At the upper site a detailed study was made of the buried gorge to aid in designing the dam. The depth of overburden was also determined along the line of a proposed tunnel. At the lower dam site the buried gorge was studied over a length of 1 mile, since it was also desired to gather information bearing on the feasibility of excavating a long tail-race channel.

Enough was known concerning the rock and overburden to give a reasonable expectation that the Schlumberger method could be successfully applied. Similar schists studied electrically at other places in Canada and New England had shown a high resistivity, around 3,000 to 4,000 ohms per meter cubed, as is to be expected for such dense, metamorphic rocks. The resistivity of unconsolidated formations is less, as a rule about one-eighth to one-tenth that of the schists, except when composed of dry, sandy material.

A demonstration of the method was arranged at eight pre-existing borings, during which the electrical observer was kept in ignorance of the conditions already encountered in the course of drilling. His results were checked with the drill records and found to be satisfactory, after which an extensive campaign was decided upon.

Results at Three Holes—The results of some of the determinations are described below as representative examples. The various graphs (Fig. 3) show the resistivities calculated as a function of depth for holes 1, 10 and 14. The resistivities in ohms per meter cubed are plotted as ordinates against the depth in feet to which they correspond.

The profiles obtained can be arranged in several

groups, illustrating different types of overburden, and the three mentioned were chosen, one from each group, for illustration and discussion. The simplest type is that determined at Hole No. 1, Fig. 3. The overburden consists of essentially homogeneous material with a resistance of 500 ohms or less per meter cubed. Beneath this overburden is the schist, with a much higher resistivity, of 3,000 to 4,000 ohms and more, in fresh, unaltered rock. The electrical prediction is easy to make and places the location of the rock on the rising part of the curve. In a case such as this, where the depth to bedrock is small, irregularities of either surface or bedrock topography may have a comparatively large effect on the electrical determinations.

Another example of a conductive overburden of sandy clay, but with an intercalated layer of resistant sand and gravel, is given by the determination at Hole No. 10. The profile at Hole No. 14 is of a third type. At the surface there is a sandy, resistant layer whose thickness can be determined. Next there is a clayey, more conductive series of deposits resting on the schists. This interpretation of the overburden was confirmed by the drill samples. Such an arrangement of a conductive layer between two highly resistant masses is favorable to the interpretation of the electrical observations. The problem is difficult, if not insoluble, however, when the resistant sand or gravel deposit rests directly on resistant bedrock.

Differentiation of two materials requires a ratio between their conductivities appreciably different from unity. When this is not the case the top of a resistant sand may be mistaken for the rock.

Such a difficulty arose in one place at the dam site when the electrical determinations there indicated a rock hill in the middle of the buried gorge. Geological indications clearly pointed to the improbability of such an occurrence, however, and a subsequent boring proved that the hill consisted of gravel and boulders instead of bedrock. It should be noted that the top of this gravel formation was accurately given, as will be seen below.

This demonstrates the desirability of close co-operation between the geologist and the electrical observer. Providing such a co-operation is realized, mistakes of this kind are readily detected and are unimportant.

It is interesting to learn the average accuracy in ten of the electrical determinations that were directly checked by the drill (Table I). These represent what may be expected under the disadvantageous conditions of irregular surface and bedrock topography and of discontinuous, changeable glacial deposits.

TABLE I—RESULTS OF ELECTRICAL DETERMINATIONS DIRECTLY CHECKED BY THE DRILL

Hole No.	True Depth to Rock, Ft.	Predicted Depth, Ft.	Predicted Depth in Relation to the True Depth, Per Cent
1	29	24	83
7	27	41	151
9	108	108	100
10	100	118	118
11	56	53	95
12	50	43	86
13	101	105	104
14	142	145	104
20	167	115	69
21	147	142	97

At Hole No. 7 the depth to bedrock has too high a figure. It is probable that here the drill struck a high point in the bedrock while the electrical determination included some of the surrounding area where the rock is possibly lower. The fact that this hole is known to be close to the edge of the buried gorge makes this explanation a reasonable one.

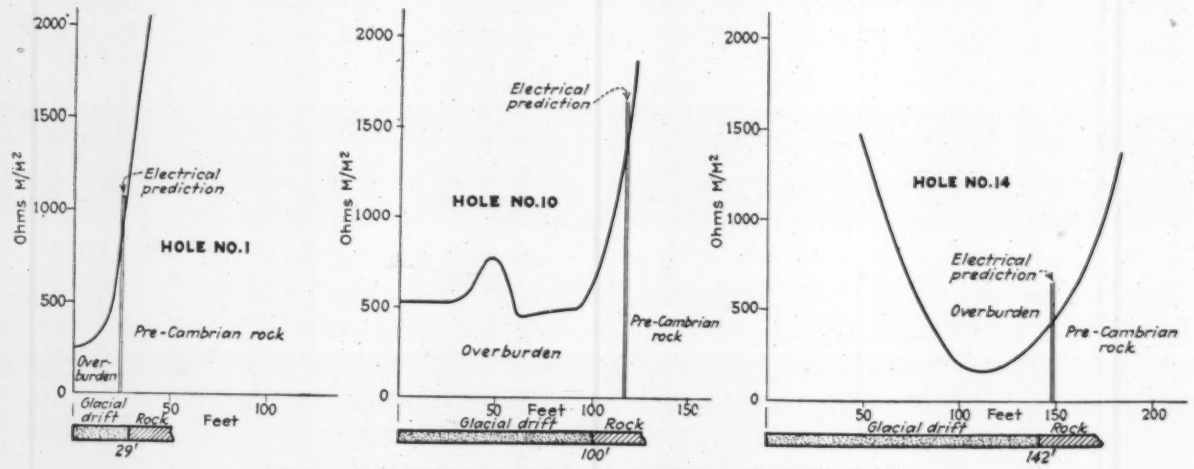


FIG. 3—ELECTRICAL AND BORING RESULTS AT THREE HOLES

Forty feet of steel casing had been left in Hole No. 10, which may have influenced the electrical results.

The gravel hill, as already mentioned, was penetrated by Hole No. 20. The prediction of 115 ft. corresponds well with the depth of 121 ft. at which the electrically highly resistant formation of gravel was actually encountered.

Following the tests an extensive exploration campaign was undertaken, as already mentioned. The results of this cannot be made public as yet, but it provides a basis for cost estimates.

Two areas, totaling 250 acres, were surveyed by establishing about 200 points of observation. This permitted the drawing up of a contour map of the surface of the bedrock in the area studied. The total cost of this survey to the construction company was not much more than that of one drill hole in which much trouble was caused by boulders. Under favorable conditions from one to three determinations a day can be made at a cost of $50 to $100 per position.

Naturally, the cost of such work will vary with the conditions encountered, such as topography, vegetation, depth of bedrock, character of overburden, and especially the detail desired in the final determinations. This latter point controls the density of the points of observation.

Other Work—Electrical exploration of dam sites has

not been confined to the two at Fifteen Mile Falls, but, for example, is now being used in the preliminary study of two sites on the upper St. Maurice River, in Quebec. It is necessary to gather information here about the depth to rock in order to complete the geological investigation and to obtain sufficient data to permit an engineering study during the winter of the various possible power developments. These sites are in wild, rugged country, and in the limited time available this information could be obtained by no other means than an electrical exploration.

Conclusions—The example cited shows the application of electrical methods to a specific foundation problem. That this technique has a wide field of use hardly requires further emphasis. It can be utilized wherever there is need of investigating rock formations covered by overburden or overlying strata, provided the electrical requirements of differing resistivities, etc., are met. Other instances that come to mind are studying foundations for buildings and bridge piers and studying the depth of overburden above tunnel lines.

Electrical prospecting will not eliminate drilling, of course, but will greatly cut its cost by concentrating such drilling only on checking the crucial conclusions of the electrical survey. In this manner the speed of such work can be increased while the cost is at the same time reduced.

The apparatus required for this work is, fortunately, very light and easily transported. It weighs only a few hundred pounds and no single piece is too heavy to be conveniently carried. Consequently it can be taken in a canoe, on a pack horse, in a Ford, or carried by two or three men. This makes it of especial value on preliminary work difficult of access where some information must be gathered prior to deciding on further development. Also, no complicated preparation of the terrain is necessary, such as boring holes, topographical surveying, etc.

The potential method is probably not the only geophysical one which can be used for the solving of civil engineering problems. However, as far as we know, it is the first to have been commercially applied in this way to any problems of importance and to have solved them satisfactorily. Moreover, the dollars and cents side of the question should not be overlooked; the use of this method involves only very small expenditures for the study of large areas, even in great detail, as even the rough idea of expenses, given above, amply demonstrates. There is little doubt that no other method of geophysical exploration can meet this requirement as effectively as can the electrical one.

* * * * * *

Geophysical Foundation Study by Explosion-Wave Method

Possibilities in Applying Methods Developed in Oil-Field Work

By A. T. Parsons

Engineer With Frank Rieber, Inc., San Francisco

GEOPHYSICAL surveys have become well established as a part of the preliminary work in developing new oil fields. Because there has been for several years a demand for studies of this sort in mining and oil prospecting, apparatus has been developed and improved to a high degree of sensitiveness and accuracy. With experience in the interpretation of the results obtained with this apparatus, valuable information may be had about hidden geological structures. The relatively low cost and the comprehensive scope of geophysical surveys as compared with investigations made by boring may make them worth considering as aids in foundation explorations at proposed sites for dams, bridges or buildings.

The first question the designer of foundations must ask is, "How far down is it to something solid?" Among the geophysical methods now in use, those employing one or another modification of the seismograph can help to answer this question.

The Explosion-Wave Method—In such work a sensitive receiving device called a receptor is placed a few feet below the surface and connected with equipment arranged so that a record is obtained of the time required for the energy sent out by an explosion to travel from the point of the explosion to the receptor. Details of practice vary with the different organizations making the survey.

Fig. 1 shows the type of receptor used in our own work. A heavy mass and a crystal are so arranged that the slightest movement of the earth generates piezo-electric energy. Amplification is provided, such that it is theoretically possible to record an earth movement equal to the diameter of a hydrogen molecule; practically such extreme sensitivity is not required. The energy is transmitted to a truck in which are mounted the photographic recording equipment, dark room, etc.

Communication between the operator at the instrument truck and the powderman is maintained by a telephone line usually laid on the ground, which is also used to fire the charge. When the charge has been laid and is ready to fire, the powderman signals to the operator. When both are ready, which includes the powderman and his equipment being at a place of safety, the powderman closes a relay. The operator then makes final adjustments and presses a key that fires the charge, but he cannot do this until the powderman has already closed his relay.

Photographic Recording—The record, obtained photographically, shows the shadow of three wires on a film traveling at a controlled rate of speed. Fig. 2 is a copy of a typical record. The upper wire is actuated by a circuit which includes a wire wrapped around a stick of dynamite in the charge. At the instant of explosion the circuit is broken, and the wire is violently agitated as at *A*, coming to rest immediately afterward. This is a more accurate method of recording the instant of explosion than one depending directly on the firing of the cap, since caps have a variable lag of a few thousandths of a second.

The middle wire is actuated by the mechanical energy of the explosion wave being transformed into electric energy by the crystal in the receptor. At *B* the wire starts vibrating through a wide amplitude as a result of the earth waves set up by the explosion reaching the receptor. *B* thus marks the instant of arrival just as *A* marks the instant of explosion.

The lower wire is actuated by a 250-cycle tuning-fork, the distance between two successive sawteeth corresponding, therefore, to 0.004 second. Using it to measure the

time between instant of explosion and instant of arrival of the first wave at the receptor, the time of wave travel from the point of explosion to the receptor may be read with accuracy to the nearest 0.001 second. In this particular instance the distance from explosion to receptor was 2,500 ft., and the travel time 0.430 second.

Note in Fig. 2 the small vibrations on the middle wire before the first wave from the explosion arrives. These may be the result of traffic on a near-by highway, of wind in trees, of "microseisms" or minor movements of the earth's crust itself, or of one or more of a number of other causes. Care in planning the work and in the various details of field operations will reduce but cannot entirely eliminate such "unrest." This is the reason for practical limitation of sensitivity. With extreme possible amplification the movements of unrest might be so large as to make it impossible to distinguish between them and the first arriving explosion wave.

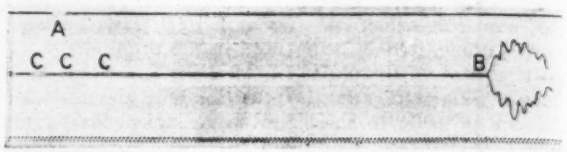


FIG. 2—TYPICAL TIME-TRAVEL RECORD
A corresponds to the instant of explosion and *B* to the instant of wave arrival at the receptor. Disturbances such as *C* are the result of some form of unrest. The tuning-fork time record is below.

In practice, the operator adjusts his sensitivity to give an approximately quiet wire. Roughly speaking, when there are no special causes of unrest, a sensitivity such that a light tap with the foot on the ground a hundred yards away from the receptor will affect the wire will give the most satisfactory records. In oilfield work it is important, on account of cost of explosives, to maintain the maximum practicable sensitivity.

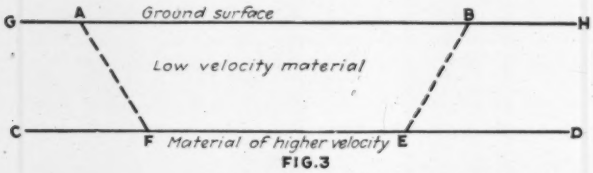


FIG.3

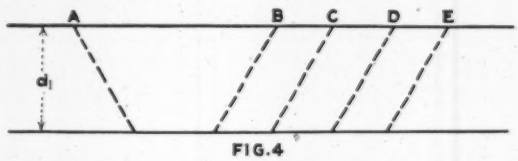


FIG.4

FIGS 3 AND 4—EXPLOSION-WAVE PATHS FROM SINGLE AND SUCCESSIVE SHOTS

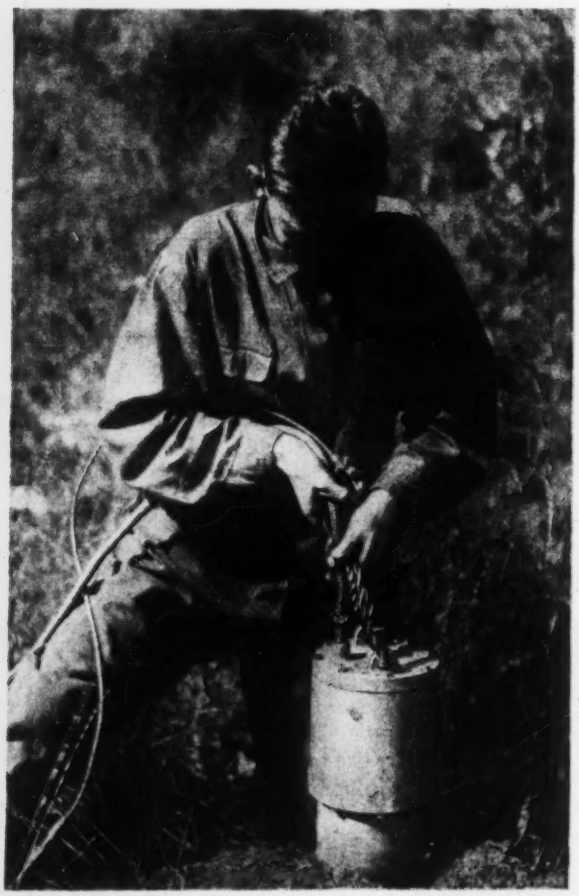


FIG. 1—LOWERING THE RECEPTOR INTO THE HOLE
The shielded cable on the left leads to the instrument truck.

If, now, a receptor is placed at point *A*, Fig. 3, and a charge of explosive is fired at point *B*, energy will be sent out in all directions. Certain waves travel to the receptor in a practically straight path close to the surface. Others travel downward at various angles with the horizontal. These will ultimately strike a lower stratum at various points on the interface *CD*. This lower stratum will usually be more elastic and will therefore permit a greater velocity of wave travel. Certain of these waves will strike the interface *CD* at the angle of grazing incidence and will then be refracted along the interface. Some of these waves will continually be rising toward the surface, until finally some energy will follow the path *FA* and reach the receptor.

It is evident that if the distance *AB* is small, the first arriving waves to reach the receptor at *A* from the explosion at *B* will be those following the straight path *AB*. If, however, the distance *AB* is increased, a point will be reached where, on account of the greater rate of travel along the interface *CD*, the wave following the path *BEFA* will arrive at *A* simultaneously with the wave following the straight path *AB*.

Now, if charges are fired successively at different distances from the receptor, as at *B*, *C*, *D*, *E*, in Fig. 4, and times of arrival of first waves from each shot recorded, it is possible to determine V_1, the velocity of wave transmission in the upper layer; V_2, the velocity of wave transmission in the lower layer, and L_1, the distance from the receptor at which a shot would give simultaneous times of arrival. It can be demonstrated that where d_1 is the depth to the interface

$$d_1 \sin \alpha = L_1 \frac{L_1}{2} \text{ versin } \alpha$$

where α equals the angle of grazing incidence; and by Snell's law of refraction

$$\cos \alpha = V_1 \frac{V_1}{V_2}$$

This formula for depth is accurate for level interfaces only. For inclined or irregular interfaces various corrections have been developed. Similarly, formulas have been developed for determining successive

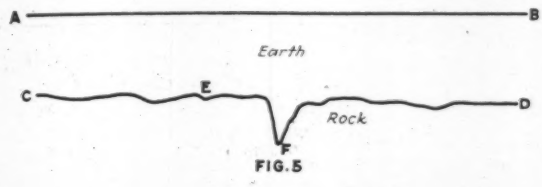


FIG. 5

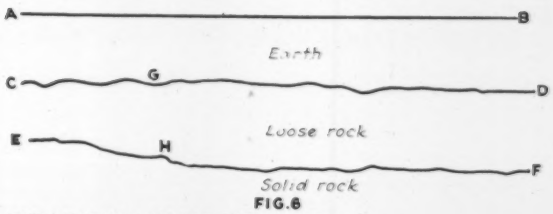


FIG. 6

FIGS. 5 AND 6—TWO KINDS OF UNDERGROUND CONDITIONS

depths to two or more interfaces. During the past two years the writer's organization has made subsurface surveys of several hundred square miles in various parts of California. Where it has been possible to compare the results with actual drilling, an essentially close agreement has been observed.

Applying the above to foundation surveys, Figs. 3 and 4 may represent ideal cross-sections of the subsoil at a dam or bridge site, while Figs. 5 and 6 represent typical cross-sections of the more usual type of conditions at the average site for a dam or a bridge. In Fig. 5, the velocity through the earth or loose material above *CD* would be materially less than that of the underlying rock.

By running a short line of shots, a depth could be determined for some point *E*. Each additional series of shots could give another depth determination. Such depths would be nearly if not quite as accurate as those obtained by diamond drilling. But in cases where old watercourses or other fissures filled with loose material occur, the explosion method would be the more dependable of the two. Almost every engineer can recall occasions when drill holes straddled such a channel and nobody knew anything about it until actual excavation exposed the bedrock. In the case of a geophysical survey, the presence of such a deep fissure could hardly fail to be indicated by some of the time-travel records, although it might be impracticable to get more than an approximate idea as to its position or extent.

In Fig. 6 the loose rock below *CD* would have a higher velocity than the earth but very much less than the solid rock below it. A depth from the surface to *G* and from the surface to *H* could both be determined.

It might be added that the engineer cannot expect to identify a particular kind of rock by the velocity alone. Weathering, degree of consolidation and a number of other factors cause the velocity of even the same continuous stratum to vary considerably. Generally speaking, however, ordinary dry soil or other dry unconsolidated material will permit a velocity of 1,500 to 2,500 ft. per second. When completely saturated, this same material will have a velocity of 5,000 to 6,000. We have found certain shales and sandstones that had a velocity ranging from 3,500 to 5,000 when dry. Below the plane of saturation, most sedimentaries appear to have velocities between 7,000 and 9,000. The volcanics, except certain kinds of lava, and the plutonics will run considerably higher, granite sometimes having a velocity of 20,000 ft. per second.

Regarding the fundamental question of cost, the data available are only approximate. In prospecting for geological structures favorable to oil, we frequently correlate strata lying 2,000 ft. or more below the surface. This may require lines as much as 16,000 ft. long and single powder charges as great as 2,000 lb. In such work, findings comparable in accuracy to those obtained by core-drilling can be obtained at a small fraction of the cost and an even smaller fraction of the time required for the latter work.

Foundation surveys present radically different requirements and conditions of operation. Depths greater than 100 ft. or so would not be of interest. Lines would therefore seldom exceed 1,000 ft. in length, and the maximum powder charge would be a pound or less. The cost of explosives, an important item in oilfield work, would therefore be negligible in foundation surveys. Other items affecting cost would also be materially reduced. On the other hand, of course, shallow core-drilling is cheaper per foot than deep core-drilling. Nevertheless there is good reason to expect that equipment and technique can be developed such that foundation surveys could be made by geophysical methods at a materially lower cost than by core drilling.

Spillway Cut Rushed to Safeguard Lake Pleasant Dam

High Multiple Arch in Trouble—Long Cracks, in Buttresses, Excessive Stresses and Other Defects—Numerous Reports

A SPILLWAY channel 24 ft. lower than the lip of the existing spillway of the recently completed Lake Pleasant multiple-arch dam in Arizona is being excavated with the utmost dispatch as an emergency measure to safeguard life and property below the dam. A flood in the Agua Fria River of a size that might occur at this season of the year could fill the Lake Pleasant reservoir in less than two days; the dam has been declared unsafe for the load imposed by a full reservoir, and since the capacity of the three 48-in. outlets through the structure would afford but negligible relief in a major flood a serious situation has arisen.

As the district owning the dam was unable to finance emergency work which engineering boards found necessary, a campaign for funds has been made among various interests affected and $40,000 was raised up to Jan. 28. At that date work had already been under way for about a week, the spillway gates had been taken out, the concrete of the spillway was being removed and two steam shovels and a dragline were at work on excavation, with other equipment en route to the job. It was expected that within 30 days the new outlet, with a capacity of 150,000 sec.-ft., below the original spillway would be completed. A bill calling for an emergency appropriation of $150,000 to reimburse contributors to this emergency fund and do other necessary work on the dam has been introduced in the state legislature and favorably passed by committee.

Although the Lake Pleasant dam is only 170 ft. high above streambed, the depth of its concrete cutoff walls gives it a total height of 258 ft., which is a record for structures of this type. It was completed in September, 1927, for Maricopa County Water Conservation District No. 1; design and construction were described in *Engi-*

neering News-Record of Feb. 2, 1928, p. 180. Announcement of investigation of the dam by state officials appeared Jan. 17, 1929, p. 116.

Although the situation was not made public until recently, the structural safety of the dam was questioned two years ago. In February, 1927, when cracks developed in the buttresses while the dam was under construction, Fred A. Noetzli, consulting engineer, Los Angeles, was called in by the contractor to analyze stresses in the dam. As a result of this analysis Mr. Noetzli urgently recommended that the design be changed to strengthen the structure, particularly by the addition of reinforcing steel in the buttresses. (Mr. Noetzli's findings are given in more detail later in this article.)

Official Investigation—Last summer state authorities investigated conditions at the dam, when criticism developing in various quarters began to focus expert attention upon the structure. Further stress analyses and several engineering reports were made. A collection made in Phoenix on Jan. 3 included reports by the following: C. C. Cragin, general superintendent and chief engineer, Salt River Valley Water Users' Association; Fred A. Noetzli, consulting engineer; J. A. Fraps, engineer, Arizona State Certification Board; J. H. Manning, vice-president, Stone & Webster, Inc.; B. F. Jakobsen, consulting engineer; and a consulting board consisting of J. B. Lippincott, D. C. Henny and F. L. Ransome.

On Jan. 9, 1929, the state water commissioner appointed a general committee of thirteen, representing various interests involved, including the two transcontinental railroads crossing the valley below the dam, the telephone company and water districts, for the purpose of making a general survey of the situation and reaching some agreement about remedial measures. This committee met on Jan. 14 under the chairmanship of W. W. Lane, state engineer, and was grouped in subcommittees for the investigation of (1) structural conditions of the dam, (2) hydrology, (3) probable damage in areas below if the dam failed under load and (4) the best means of keeping reservoir level below the elevation found permissible.

The subcommittee on hydrology reported that the drainage basin contains 1,500 square miles, for which spillway provision should be made on the basis of a flood runoff of 160 sec.-ft. per square mile, or 157,500 sec.-ft. at the dam; that there should be a freeboard of at least 3 ft. above maximum water level at such a flood; and that during the eighteen-year period for which runoff records are available the level of the stored water would have risen above the spillway lip six times. Although the three needle valves installed in the dam afford only negligible relief in a major flood, if four temporary openings were to be cut through the dam, each 25 ft. square, they would pass a flood of 100,000 sec.-ft. while the reservoir level was kept below the 71-ft. level, or would pass 120,000 sec.-ft. before the level would rise to 130 ft.

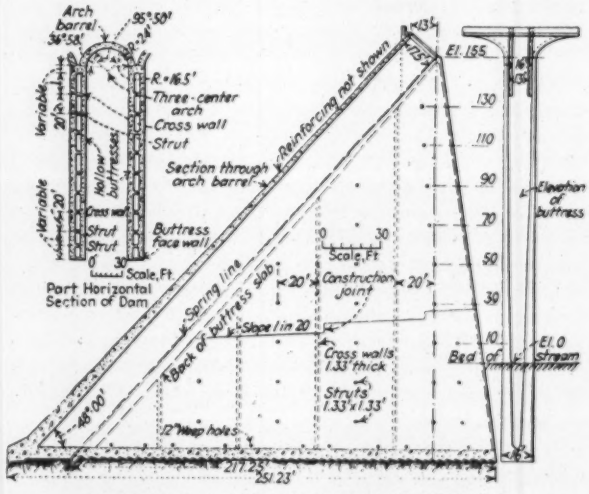
The report of the general committee found that the dam is not safe under full reservoir pressure and that "the water in the reservoir cannot be held down to a safe elevation in case of major floods without providing an additional flood outlet." Since water level at 158 ft. would produce tensile stresses in the buttresses not exceeding about 60 lb. per sq.in., this level was adopted as the maximum permissible reservoir stage, pending final strengthening of the dam. In view of the possibility that a flood might occur before this strengthening could be effected, it was decided that a cut in the present spillway should be made, 24 ft. deep and 175 ft. wide at the bottom.

This involves destroying part of the concrete structure supporting tainter gates in the existing spillway, whose lip is at El. 154 above streambed, and digging a cut that would lower the outlet to El. 130. This location for an emergency outlet was selected because it offered the quickest way of letting floodwaters out of the reservoir other than making openings in the dam itself. It was believed that the excavation, about 400 ft. long and involving about 95,000 cu.yd., could be completed within 30 days by operating steam shovels on a three-shift basis. Contract for the excavation was awarded the Sharp & Fellows Contracting Company, Los Angeles.

This work was not undertaken, however, until officials of the district which owns the dam passed a resolution approving the deepening of the spillway and releasing contributors to the emergency fund from all liability that might arise therefrom.

The general committee also recommended to the state water commissioner that if it appears, as this 24-ft. cut nears completion, that the dam is likely to remain in its present condition for a year or more, the cut should be deepened 10 ft. more; that "the weakness introduced by cracks in the abutments be corrected in the best and quickest manner which can be devised," and that in case the cracks should lengthen seriously or "in case of an immediate emergency the executive committee be authorized to act according to its best judgment." The executive committee is a group of four Phoenix engineers, headed by the state engineer, appointed to act in the matter for the state water commissioner in any new emergency that might arise. The personnel was suggested by the general committee of thirteen and appointment was made accordingly by the water commissioner.

With reference to repair of the cracks and final strengthening of the dam, the committee of thirteen recommended "that immediate efforts be made to cure the effect of existing cracks by the insertion of heavy steel shear plates, beginning with the cracks deemed most



LAKE PLEASANT DAM IN CROSS-SECTION

(From *Engineering News-Record* of Feb. 2, 1928, p. 182)

The dam is located on the Agua Fria River, Arizona, about 25 miles northwest of Phoenix. It is of the multiple-arch type, but has hollow or double-wall buttresses. The buttresses are spaced 60 ft. center to center, and each is 16 ft. wide out to out of walls, so that the arches have a clear span of 44 ft. The springing-line slope is 10 on 9, and the batter of the downstream edge of the buttresses is 0.15 to 1. The foundation is at some buttresses considerably deeper than that shown, one buttress being carried down to 81 ft. below streambed. In another part of the dam a cutoff wall at the toe extends to almost the same maximum depth.

injurious, and that if cracks in the buttresses should extend into the bases of the buttresses or should interconnect to a material extent or should break through to the top, the injurious effect thereof be studied, and that in addition stress calculations be promptly checked and that safe elevations before mentioned be corrected and width and depth of the new spillway channel be amended in accordance therewith."

Design of the Dam—Mr. Noetzli's analysis of stresses in the dam was the first complete and independent check

COMPARISON OF SOME HIGH MULTIPLE-ARCH DAMS*

	Height Ft.	Inclination of Water Slope From Horizontal (Deg.)	Length of Buttress Base in Ft. 50	100	150	200
			Below High Water Level			
Lake Pleasant, Ariz.	256	48	38.75	91.25	143.75	196.25
Tirso dam, Italy	239	57	63.0	113	163	213
Pavana, Italy	177	57	54.0	107.5	161	.....
Palmdale dam, Calif.	175	45	48.0	118	200	225 (175)
Tidone dam, Italy	171	45	73.5	136	198.5	.....
Florence Lake, Calif.	160	50	60.0	115	165	.....
Anyox dam, B.C.	156	51	44.0	118	170	.....
Mt. Dell dam, Utah	150	50	53.6	112.5	159.5	.....
Lake Hodges, Calif.	136	45	48.0	108.50	.....	.....
Voehrenback, Germany	95	53	45.0	95	.....	.....
Selune dam, France	50	46	75.0	.....	.....	.....

*From the Lippincott-Henny-Ransome report.

of the safety and stability of the original design, so far as is known. He found that even then (February, 1927), before the structure had been carried to the full height, "most of the buttresses had developed vertical cracks extending from the surface of the ground to some distance above, in some cases almost through the whole height of the buttresses as constructed at that time" (about 90 ft. above streambed). These cracks, described as "a most serious menace to the safety of the dam," were caused by shrinkage of the concrete and temperature variations, Mr. Noetzli said, and might have been prevented by proper use of reinforcing. The design also was pronounced "rather bold in several respects," notably in the unusually high sliding factor and in the existence of very high principal stresses.

Mr. Noetzli found that the shape of the crest "is such that the upper end of the arched barrel is cut off and the closure against the water is effected by inclined slabs between the arches. Furthermore, a wave cope 3.3 ft. high was provided, and storage level of the reservoir is only 12 in. below the top of the wave cope and therefore 2.3 ft. above the theoretical crest of the arched barrels. With this arrangement of the crest of the dam the buttress walls are relatively short in an upstream and downstream direction, especially in the upper part of the dam."

After analyses of stresses, he pointed out that in the design of the dam "important facts apparently overlooked were as follows:

"1. The bearing pressure of the arches upon the buttress walls is very high, which fact, in connection with the relatively short length of the buttress walls, produces excessively high principal stresses at the upstream ends of the buttress walls.

"2. The reinforcement of the buttress walls is entirely inadequate. The omission of any horizontal steel in the buttresses probably is largely responsible for the cracks which have developed . . .

"3. The sliding factor in the upper portions of the dam is quite unusually high, being in excess of 0.80 at all elevations above +50. Anything in excess of 0.80 would appear to be too bold at our present state of knowledge relative to this type of structure."

If the buttresses had remained intact, they probably would have been strong enough as designed, he found, in spite of having but little reinforcing. However, "the occurrence of vertical cracks evidently changed the statical condition of the buttresses to such an extent as seriously to endanger the stability of the dam." Even without cracks a stress analysis showed full load would produce maximum inclined tension stresses at the upstream side of the buttress walls of almost 200 lb. per sq.in.

On Mr. Noetzli's recommendation additional steel was put in those parts of the buttresses near the top that had not been completed when his report was rendered. Nothing was done, however, to strengthen the lower portion of the buttresses which he found to have inadequate steel.

Another opinion on design, stressing features that need attention and, in general, in conformity with the Noetzli findings, appears in the Lippincott-Henny-Ransome report, already mentioned. This consulting board was retained by the district owning the dam, and its report, dated Dec. 15, 1928, found the dam to differ in important particulars from other similar structures. The most important differences in respect to stability and safety, the report finds, are the upstream slope of the dam and the relation of the length of buttress walls to depth below high-water elevation. (An accompanying table gives a comparison of these elements in various dams of this type.) The Lake Pleasant dam is not only the highest of its type but, "judging from the main stability features, the boldest, mainly because of shortness of buttress walls."

The concrete was found of fair quality, and the design was pronounced safe so far as compressive stresses in the buttresses are concerned. "There are, however, tensile stresses in the concrete next to the groin of the arch far higher than can be carried by the small amount of steel provided, especially in the upper part of the buttresses, and full loading will probably produce inclined cracks."

Shrinkage cracks were found in all but the lowest buttresses, which was regarded as "an element of serious danger" . . . "The direction of the cracks is generally vertical and they have opened up from a point of mere visibility to a width of 0.05 in. and one crack in buttress 4 measures 0.1 in. Near the upstream end of the buttress walls the cracks tend to become normal to the water slope. The crack in one of the two walls composing the buttress is usually accompanied by a corresponding crack in the other wall, although the position is not always directly opposite and in a few instances only one wall shows a crack." With buttress walls cracked and loads transmitted to the foundation in a manner not intended in the design, there is a redistribution of stresses and intensified loads on the concrete and in the foundation. Loads on the concrete in the highest buttresses might thus increase to more than 2,000 lb. per sq.in. "It is apparent that the concrete in the walls may become overstressed and cause settlement and failure, even assuming the foundation to be perfect."

As to permissible foundation loads, the board believed that the maximum loading of 8.6 tons per square foot contemplated in the original design should not be greatly exceeded in any design for the permanent strengthening of the dam.

Among its conclusions this board finds that (1) "the dam is unsafe for overflow and with the present spillway

the dam may be overtopped"; (2) "the dam as it stands has satisfactorily resisted a water load of 117 ft. and is probably safe at somewhat higher levels, say 125 or 130 ft., with the possibility that it may stand a load with water at 154 ft. provided cracks do not extend"; and (3) "it is practicable from an engineering standpoint to make the dam safe for any desired water elevation not exceeding 168 ft."

Geology—The geological formation at the site is described in the Lippincott-Henny-Ransome report as volcanic rocks of Tertiary age, associated with imperfectly cemented gravels. The rocks include basalt, tuff, tuffaceous sandstone and conglomerate. "A notable feature of all these rocks, and a decidedly unsatisfactory feature from an engineering point of view, is the irregularity of their distribution. Layers of tuff may be thick in one place and thin in another," and "absence of persistent regular stratification makes it impossible to predict from any particular drill holes what may be found in another boring only a few feet away."

After describing the varying character of the tuff this report says: "Obviously no buttress should rest entirely or even in part directly on the soft variety of the tuff. All available information indicates that buttresses 16, 17, 18 and 19, which rest on tuff, are founded on the harder variety of that rock. The tuff as a whole has low resistance to erosion, as the softer varieties are readily removed by running water and the harder portions are thus in turn laid open to attack, particularly by undermining."

Buttresses Nos. 1 to 14, inclusive, rest on basalt; buttresses 15 to 19 rest on tuff which under this part of the dam fills a depression in the basalt about 90 ft. deep. The cutoff wall here is reported to have been carried down to basalt, but the buttresses rest on tuff. Buttresses 20 to 27 rest on basalt.

The result of the geological study, the report states, "indicates that the foundations, although far from ideal, are adequate for a dam of conservative design having a height of the present structure and having the load properly distributed on the foundation. The character of the foundation rocks, however, is not such as to justify any venturesome departure from a design known to be safe on a good foundation. In other words, Lake Pleasant site is no place for experiments in dam construction."

Railway Construction in 1928

New railway lines and extensions built during 1928 had a total length of 1,025 miles, with 1,342 miles of track, according to figures given in *Railway Age*. This compares with 779 miles of new line in 1927 and 1,005 miles in 1926, and was the highest record since 1916, with 1,098 miles. This construction was in 35 states, of which Texas was far in the lead with 353 miles, followed by Montana, 163 miles; Illinois, 127; Alabama, 124 miles, and then Wyoming, with 60 miles. Second track totaled 271 miles, as against 446 miles in 1927; third and fourth tracks were only 25 and 20 miles respectively, as against 18 and 10 miles in 1927. Lines abandoned in 1928 aggregated 512 miles, so that the net addition to the railway system was 513 miles. In 1927 and 1926 the abandonment amounted to 282 and 457 miles, respectively. Railway construction in Canada in 1928 ran up to 723 miles, by far the highest record since the 1,978 miles in 1914. Only 11 miles was abandoned. Mexico built only 58 miles.

Raising of Aswan Dam in Egypt Proposed

International Board of Engineers Recommends Increasing Height of Structure by 30 Ft.

THE second raising of the crest elevation of Aswan dam on the Nile River in Egypt has been recommended to the Egyptian government by an international technical commission composed of Hugh L. Cooper, New York, W. J. E. Binnie, England, and H. E. Gruner, Switzerland. The commission's report was made to the Egyptian government in December, as noted in our issue of Jan. 3, 1929, p. 37. Its findings in regard to the manner in which the height of the dam should be increased were unanimous, the first time in the history of the construction and enlargement of that great structure that any board or commission has rendered a unanimous decision. The findings are now being considered by the Egyptian government, and it is confidently expected that the work will be undertaken in the near future.

Construction of the Aswan dam was begun in 1898 and the work was completed in 1902. The structure as then built was about 6,400 ft. long, consisting of a solid masonry section 1,800 ft. long and a section 4,600 ft. long containing 180 sluiceways and a lock at the west bank. The greatest height above foundations was about 96 ft. and the maximum depth of water in the reservoir 66 ft. The dam was constructed of granite rubble masonry laid in portland cement mortar and faced with square rock-faced granite laid in courses varying from 12 to 24 in. in thickness.

In 1907 a contract was let for raising the height of the dam 5 m. (16.4 ft.). The work was done by adding masonry 5 m. thick to the downstream face of the dam, using a scheme developed by the late Sir Benjamin Baker to overcome the temperature stress difficulties which would develop in attempting to join the old and new masonry. Tie-rods $1\frac{1}{4}$ in. in diameter and $8\frac{1}{4}$ ft. long were sunk into the surface of the old masonry so as to project about half their length. These rods were set in rows about 1 m. apart, both horizontally and vertically. The new masonry was then built over the face of the dam, leaving a space of $\frac{1}{2}$ ft., measured normal to the face, between the new and the old masonry, except for small supporting walls which were built 49 ft. apart. These walls and the tie-rods supported the new masonry until the masonry on each side of the opening had reached the same temperature, when the space between the old and the new masonry was filled with grout.

With this increase in the height of the dam of 5 m. the water surface elevation was increased 7 m.

Present Condition of the Dam—The first undertaking by the technical commission was an examination of the existing dam to determine how effectively the masonry added in 1907 had bonded with the older masonry to form a monolith, also to examine the entire structure as to soundness. Thirty-six minor vertical cracks were found in the upstream face, starting from the arch over the sluiceways and extending from one to six courses above the opening in the pierced section of the dam and starting at the bottom and extending upward for a maximum of ten courses in the solid section. On the downstream face, 175 small vertical cracks in the pierced section and about 27 vertical cracks in the solid portion

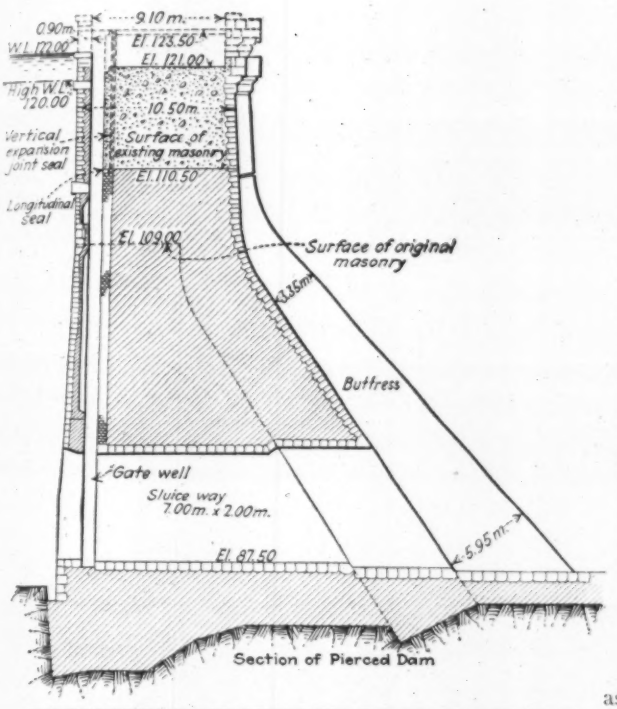
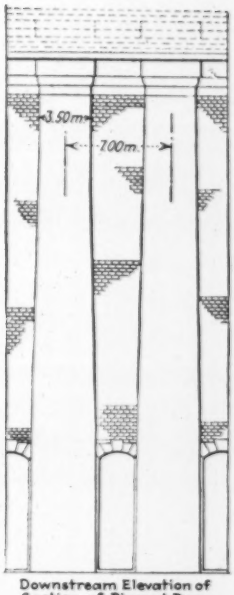


FIG. 1—PROPOSED RAISING OF ASWAN DAM



were examined. All the cracks in the pierced section originated in the crown of the sluiceway arches and extended upward more or less vertically for distances of about 3 m. in most instances. Some of these cracks were only surface cracks, but others were traceable as far back as the joint between the old and the new masonry. These cracks were never more than 1½ mm. in width, and, according to the resident engineers on the dam, have not been moisture bearing except in the solid section of the dam, where the flow has never been appreciable. There were found, however, many places on the downstream face of the dam where very small discharges of water have occurred at isolated spots, showing conclusively that the existing work is permeable and permits the travel of water in voids and in cracks in the existing masonry.

Examination of the sluices showed evidence of a small amount of water finding its way through invisible cracks to the vertical walls and roof of the sluiceways. Examination of the bond between the old and the new work was made by cutting away along the joints on the sides and roof of each sluiceway to a depth of about a centimeter and by a surface examination of the joint in the floor. Each member of the commission examined these joints independently and all came to the conclusion that the scheme devised by Sir Benjamin Baker had created an absolute monolithic condition which had continued during the sixteen years that have elapsed since the work was completed. Examination of the aprons below the sluiceways which had been constructed after deep erosion had occurred in the rock below the dam in the early years following its construction showed the apron to be in excellent condition, no repairs having been necessary since 1916, and it is able to withstand increased velocities and volumes. No evidence of upward pressure on the apron was found. The board recommends, however, that further examination be made during the excavation in the apron for the proposed buttresses, and that if evidence of underflow is found grouting with portland cement be undertaken. Examination of the rock immediately adjacent to the dam indicates that the rock under the dam is adequate to carry high compressive stress.

Designs for Raising the Dam—The commission was asked by the Egyptian government to examine schemes for increasing the height of the dam which had been prepared for it and to recommend the adoption of any one, or if none was found satisfactory to recommend a scheme of its own. These schemes consisted of one prepared by Sir Murdoch MacDonald and partners, one by Prof. L. Pottérat, and three alternative schemes proposed by the Director General of Reservoirs. These latter schemes were: (*a*) A proposal to lend additional support to the portion of the existing dam below El. 114 by means of a sand-backing carried on a system of arches to permit of the discharge of water from the sluices; (*b*) a proposal to add a layer of masonry 5 m. thick to the downstream battered face of the existing structure, raising the vertical portion to the required height; and (*c*) a similar scheme, except that the thickness of the added masonry would be 7 m. and arches were introduced in the vertical portions to reduce the weight of masonry at the downstream face.

Conditions to be Fulfilled—The commission came to the conclusion that whatever scheme was adopted, it should fulfill the following conditions:

(*a*) That it should be of such design that all the stresses are determinable.

(*b*) That the design should be such that no temperature stress would be transferred from the new to the old masonry.

(*c*) That the added part should, as far as possible, relieve the existing structure of stresses brought about by changes of temperature at the downstream face of the dam.

(*d*) That all stresses in the dam after heightening shall be well below the requirements of conservative modern practice.

(*e*) That expansion joints should be provided in all new work to prevent the formation of cracks being brought about by changes of temperature.

(*f*) That the old masonry should remain in place and be disturbed as little as possible.

(g) That the design should not detract from the architectural beauty of the present structure, but should, if possible, improve it.

Of the five designs considered, the commission believes that of Sir Murdoch MacDonald approximated most nearly to the fulfillment of these conditions. It believes, however, that a somewhat different plan would give better results, and consequently it made its own recommendations as to how the work should be carried out.

The Recommended Design—The design recommended by the commission calls for raising the vertical portion of the existing dam to the stipulated height, retaining the existing width of roadway and the construction of buttresses varying in width between sluices, the buttresses projecting from the existing structure for a distance of 6 m. at El. 87.5 and diminishing to 1 m. at the level of the existing roadway. The heightened portion of the dam would be slightly battered on the downstream face and provided with false buttresses projecting so as to have the appearance of a continuation of the buttress below, a space being left between the two to allow of the expansion of the latter in a vertical direction. The existing buttresses would be strengthened by the addition of a new buttress of the same width conforming in all other particulars to those between the sluices. To prevent leakage of water in the plane of junction between the old and new work, an asphalt key is provided.

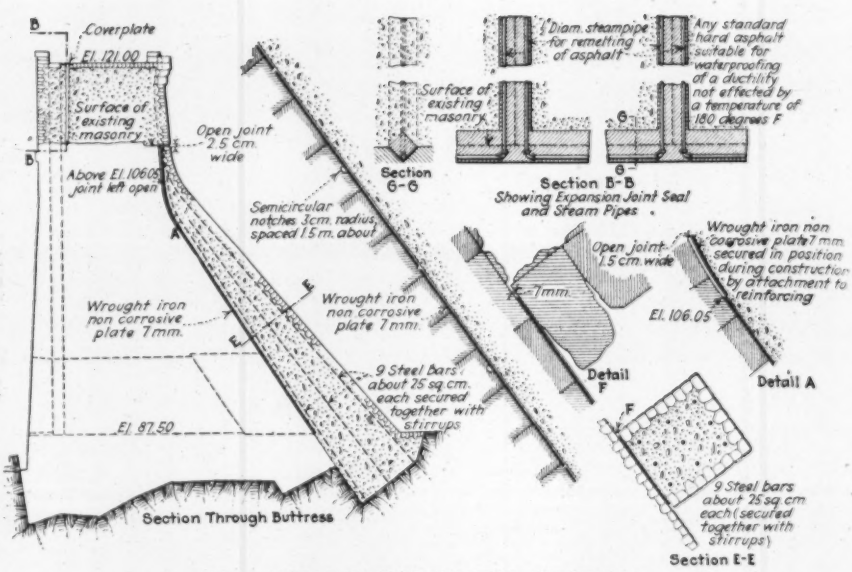

Coverplate
El. 121.00
Surface of existing masonry
Open joint 2.5 cm. wide
Above El. 106.05 joint left open
Semicircular notches 3 cm. radius spaced 1.5 m. about
Wrought iron non corrosive plate 7 mm.
Wrought iron non corrosive plate 7 mm.
9 Steel bars about 25 sq. cm. each secured together with stirrups
El. 87.50
Section Through Buttress
1" Diam. steampipe for remelting of asphalt
Any standard hard asphalt suitable for waterproofing of a ductility not effected by a temperature of 180 degrees F
Surface of existing masonry
Section B-B
Section 6-6
Showing Expansion Joint Seal and Steam Pipes
Open joint 1.5 cm. wide
7 mm.
Wrought iron non corrosive plate 7 mm. secured in position during construction by attachment to reinforcing
El. 106.05
Detail F
Detail A
9 Steel bars about 25 sq. cm. each (secured together with stirrups)
Section E-E

FIG. 2—DETAILS OF PROPOSED BUTTRESSES

The heightened portion of the dam would be divided into blocks, usually 7 m. in length, by means of transverse joints provided with vertical keys to prevent leakage, these joints being placed midway between the sluices of the pierced dam. By means of this articulation, allowance would be made for expansion and contraction of the units of the masonry without the formation of temperature cracks. A typical cross-section above El. 87.5 of the pierced section of the dam is given in Fig. 2. Examination of this figure will show that the horizontal joint between the main buttress and the false buttress will always be open at least 2 cm. at times of maximum ambient temperature.

In order to have no fixed contact between the main buttresses and the existing masonry, it is proposed that the new buttresses rest on wrought-iron non-corrosive bearing plates 7 mm. thick. The design of buttress recommended requires the dressing down of quarry faces of the existing masonry in the immediate path of the buttress for a width of 2 cm. on a side greater than the buttress width. The above plan provides for the free movement of a buttress in all directions during and after construction without the possible development of the structure's internal stresses in excess of the elastic limit of the material used.

The design proposed requires almost no disturbance of existing masonry, except the lifting of the roadway copings to the new adopted elevation. Moreover, it will protect the downstream face of the existing structure from the direct rays of the sun, thus greatly decreasing the temperature stresses in that portion over those which now exist there.

The buttresses will be independent of the present structure in the matter of movement due to temperature changes. The proposed alterations therefore will materially reduce the internal stresses which are now set up in the dam when the pool is full and the downstream face exposed to sunlight.

Greater Height Proposed—Although the commission was requested to report upon a plan for raising the dam to a height producing a top water level of El. 120, the commission believes that the work might well be continued to give an elevation of 122, in view of the moderate stresses which will be developed because of the conservative design of the older structure. The cost of this extra 2 m. of height would not exceed £300,000, whereas the amount of water would be increased from 4,585 to 5,380 million cubic meters.

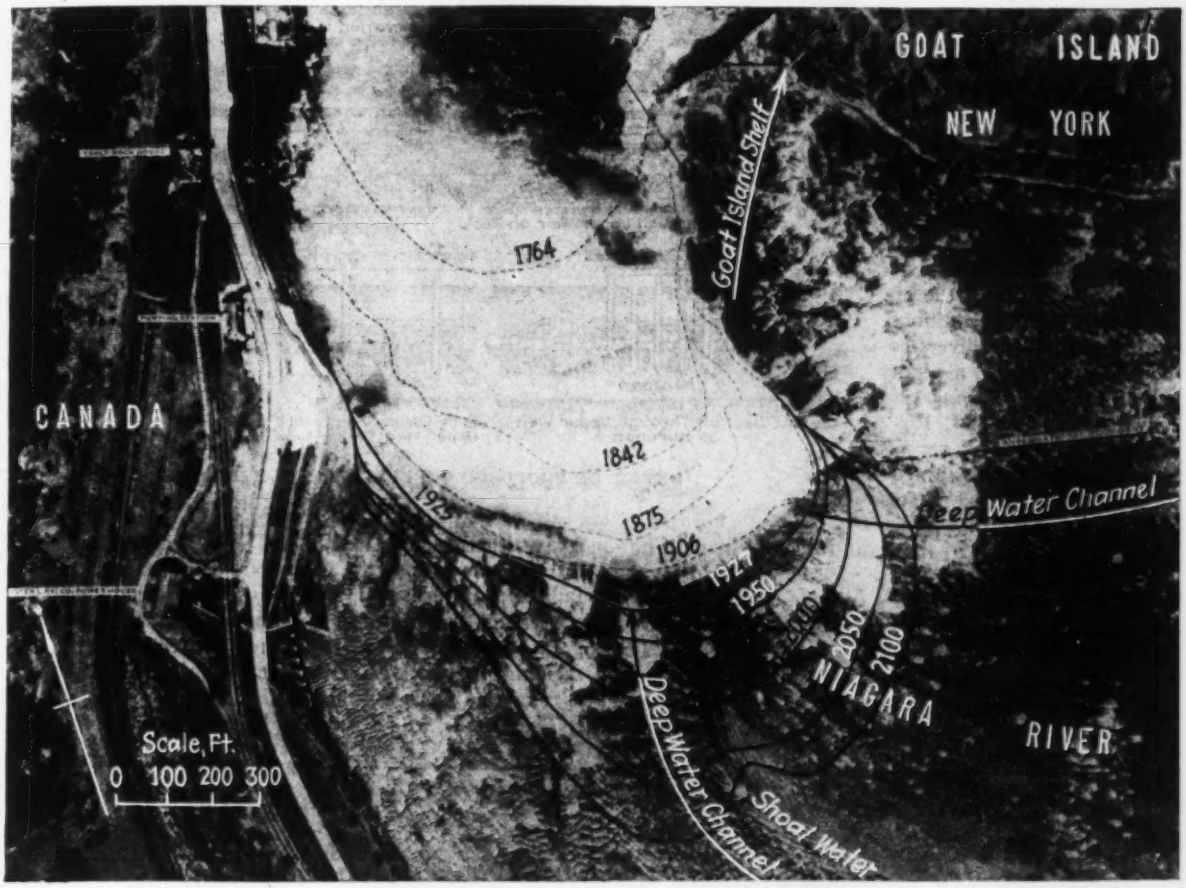
Workmanship—The commission took the opportunity to comment upon the excellence of the workmanship in the structure and to recommend to the government that the future work be intrusted to engineers and contractors who would do work of the same high standard.

Concrete Pipe Trouble at Oklahoma

Reinforced-concrete sewer pipe that failed to meet specified tests caused some trouble at Oklahoma City, Okla., last fall, in connection with the Capitol Hill storm sewer. The city's specifications do not stipulate the concrete mix to be used but require that the concrete must stand a crushing test of 3,000 lb. per sq.in. and that the pipe itself must stand a load of 2,000 lb. per foot of diameter for each linear foot of pipe. The pipes tested were 54 in. in diameter and 6 ft. long, which would require them to stand a load of 54,000 lb. Two lots of pipes were submitted by the contractor for test by the city, but both of these failed to meet the specifications as to strength and were rejected. It is stated officially that in both cases the concrete met the requirements but the pipes failed on test owing to the steel reinforcement being insufficient and improperly placed. A third lot was then made by the contractor in accordance with the requirements demonstrated by the tests. These were found to stand the specified tests and were accepted.

Changes in Flow Over Niagara Falls in Prospect

Treaty Between Great Britain and the United States Calls for Dredging and Construction of Submerged Weirs—Additional Water for Power to Be Diverted Temporarily During Winter Season—Work to Be Done by Power Company



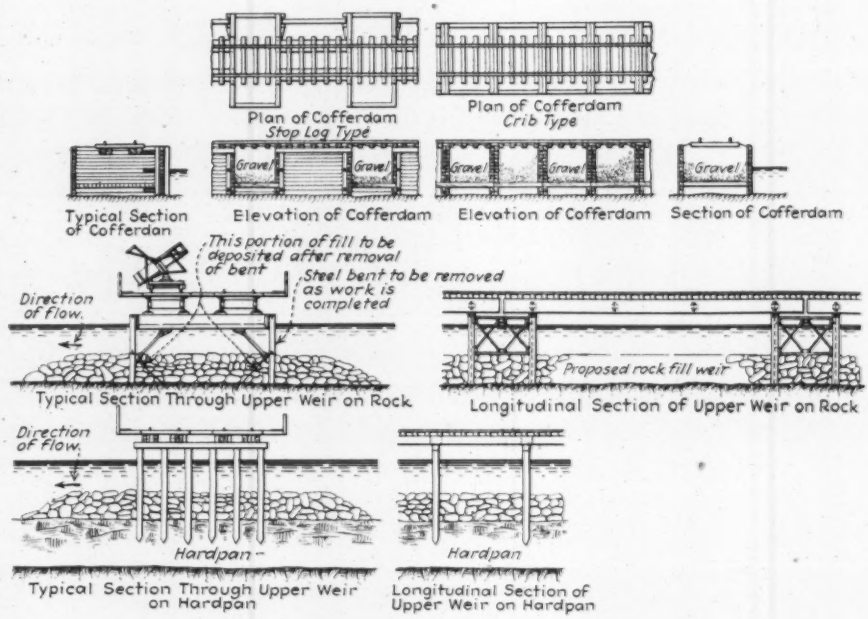
RECESSION OF NIAGARA FALLS, PAST AND PROBABLE FUTURE AMOUNTS

ON JAN. 2 a convention between the United States and Great Britain looking toward the preservation of Niagara Falls as a scenic spectacle was signed at Ottawa, Canada. This convention and its accompanying protocol are now before the Canadian Parliament and the Senate of the United States for ratification. The treaty, as written, provides for the construction of remedial works "in the Niagara River above the Niagara Falls designed to distribute the water of the river so as to insure at all seasons unbroken crest lines on both the American and the Canadian falls and an enhancement of their present scenic beauty. Concurrently with the construction and tests of the remedial work and as a temporary and experimental measure, diversion of the waters of the Niagara River above the falls from the natural course thereof additional to amount specified in Article V of the boundary water treaty of Jan. 11, 1909, may be permitted to the extent and subject to the conditions hereinafter provided: (1) The additional diversion shall be permitted only with the period beginning each year on the first day of October and ending on the 31st day of March of the following year, both dates inclusive. (2) The additional diversion to be permitted within the State of New York shall not exceed in the aggregate a daily diversion at the rate of 10,000 sec.-ft. (3) The additional diversion to be permitted within the Province of Ontario shall not exceed in the aggregate a daily diversion at the rate of 10,000 sec.-ft. (4) The provisions of this article shall terminate seven years from the date of the initial additional diversion authorized under this convention."

The protocol which accompanied the convention provides that the construction contemplated shall be carried out in accordance with the recommendations of the Special International Niagara Board along the lines of a proposal made to the board by the Niagara Falls Power Company and the Hydro Electric Power Commission of Ontario.

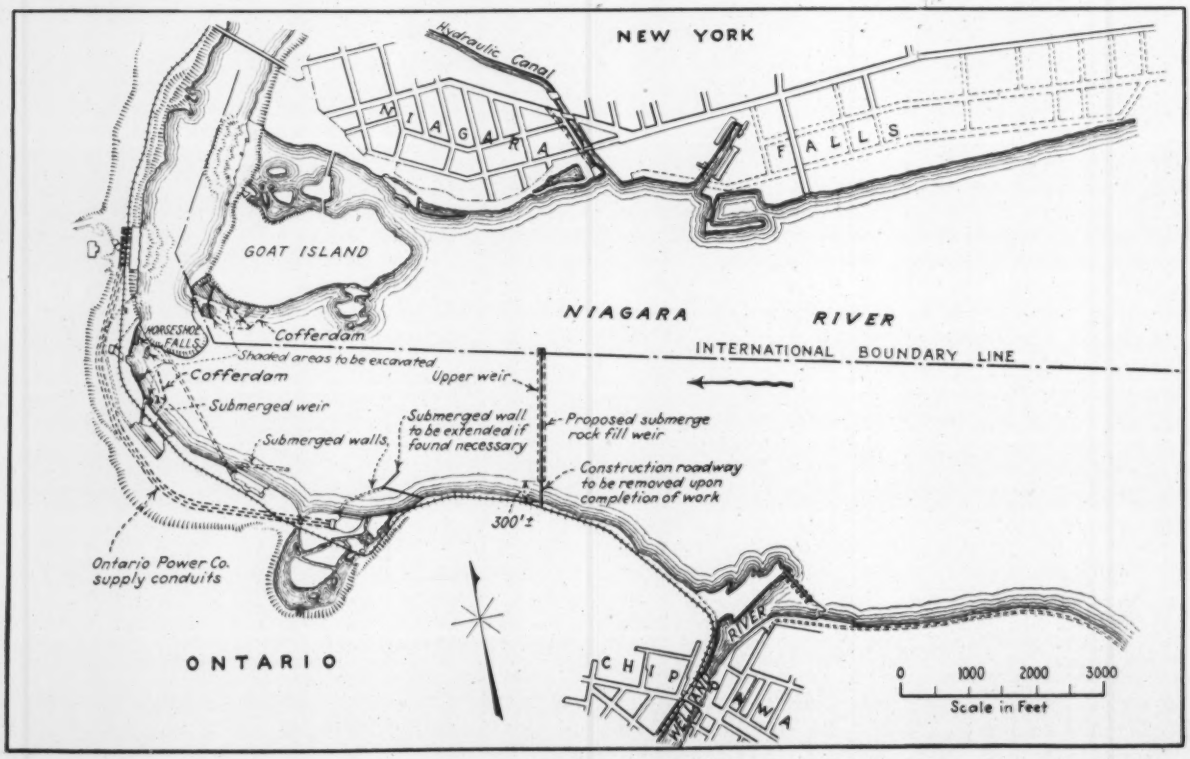
In 1925 the Canadian and United States governments set up the Special International Niagara Board to study and report upon questions relating to the Niagara River and Falls. It was asked to report upon the extent to which the scenic beauty of Niagara Falls has been and is being adversely affected by erosion or otherwise; whether the prospective impairment of the scenic beauty can be remedied or prevented, and if so, by what measures; the character, general location, sequence of construction and cost of any work required; upon the

carrying out of the proposed work, what would be the flow required to preserve the scenic beauty of the falls and river; what water might, consistently with the complete preservation of the scenic beauty of the falls and river, be permitted to be diverted from the latter temporarily or permanently; from what section of the river would it be proper to permit any diversion not already provided by the treaty, and to what extent might additional diversions be permitted in each of these sections. The board was instructed not to make a recommendation as to the apportionment of any additional water available for diversion. The board submitted a report on Dec. 14, 1927 (*Engineering News-Record*, Feb. 23, 1928, p. 332), in which it recommended early construction of works on the United States flank of the Horseshoe Falls, at the Canadian flank of the same falls and in the Chippewa-Grass Island pool. The work recommended for the flanks of the Horseshoe Falls consisted of excavations and the construction of submerged weirs for the purpose of rewatering the two flanks of the Horseshoe Falls. The works in the Chippewa-Grass Island pool were to consist of the construction of a submerged weir for the purpose of raising the level of the pool so as to throw more water against the head of Goat Island. The location and character of the proposed excavations near the crest of the falls and of the submerged weirs and walls farther upstream are shown in the accompanying drawings. Temporary cofferdams are to be built out into the river to facilitate the work of excavation. The cost of the work is estimated to be $1,750,000.


Plan of Cofferdam Stop Log Type
Plan of Cofferdam Crib Type
Typical Section of Cofferdam
Gravel
Elevation of Cofferdam
Elevation of Cofferdam
Section of Cofferdam
This portion of fill to be deposited after removal of bent
Steel bent to be removed as work is completed
Direction of flow.
Typical Section Through Upper Weir on Rock
Proposed rock fill weir
Longitudinal Section of Upper Weir on Rock
Hardpan
Typical Section Through Upper Weir on Hardpan
Longitudinal Section of Upper Weir on Hardpan

DETAIL OF PROPOSED COFFERDAMS AND WEIRS

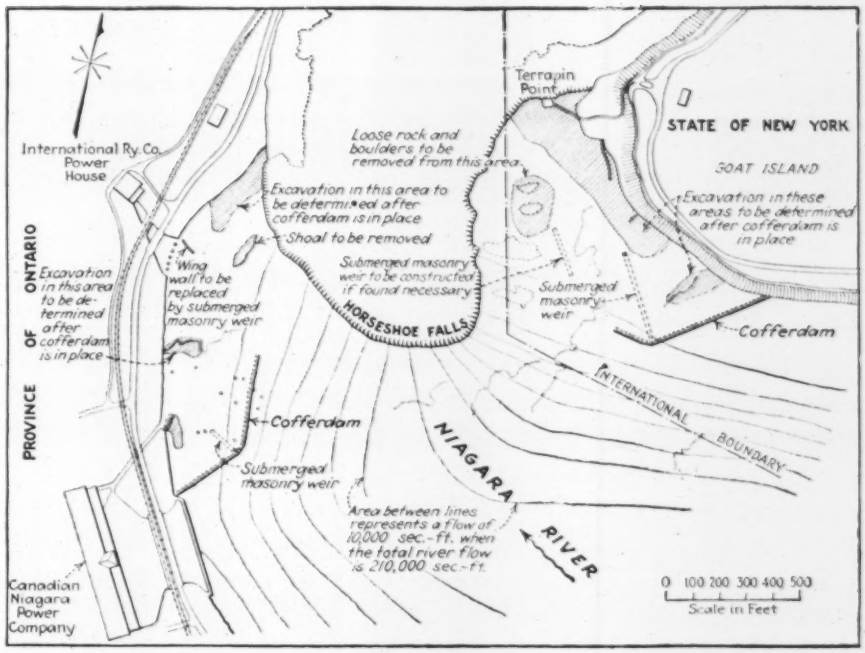
The results which the board expected from the con-



GENERAL PLAN OF PROPOSED CHANGES ABOVE NIAGARA FALLS

struction of the works on the two flanks of the Horseshoe Falls were the insurance at all seasons of an unbroken crest line from shore to shore, the maintenance of the present blended green and white color effect of the Horseshoe Falls and in a measure a modification of the rate of erosion in the bend of the Horseshoe Falls. The works in Grass Island pool were to insure an adequate flow in the American rapids and falls, and by the Three Sister Islands.

In April 1928, the Hydro Electric Power Commission of Ontario and the Niagara Falls Power Company of New York, jointly, submitted to the Special International Niagara Board drawings showing proposed works on the Niagara River calculated to conform to the recommendations of the board, and offering to construct at their own expense the initial remedial work subject to the following conditions: (1) Detailed plans, designs, method of construction and sequence of operation will be submitted to the board for its approval within three months after notice of acceptance of the proposal. (2) The board to use its best efforts to assist in obtaining all governmental authorization and the necessary permit for construction of the proposed works. (3) Construction of the proposed works on the flanks of the Horseshoe Falls will be commenced not later than 90 days after receipt by the commission and the company of the approval of the board and all other governmental authorities, and, subject to any interruptions occasioned by governmental authority, will be completed within two years after commencement, except for such reasonable extensions of time as may be granted by the board. (4) Construction of the proposed weir in the Grass Island pool will be commenced at such time as may be directed by the board after completion of work on the flanks of the Horseshoe Falls and after receipt by the commission and the company of the approval of the designs of the weir by the board and all governmental authorities, and, subject to interruptions occasioned by governmental authority, will be completed within two years after commencement except for extensions granted by the board. (5) To permit observation of the effect of the remedial work after substantial beginning shall have been made upon the work, the amount of water on each side of the river which may be diverted under the treaty shall be increased not to exceed 10,000 sec.-ft. during the season from Oct. 1 to March 31. (6) The board shall have complete supervision and control over the additional water permitted to be diverted, with power to diminish or suspend such additional diversions. (7) It is understood that the diversion for observation purposes, referred to under section 5, shall be discontinued upon six months' notice given by the government after a period of not less than ten years from the date of authorization. (8) Construction of the work shall not be considered as affecting any change in the existing ownership of or title to those parts of the bed of the Niagara River upon which they have been constructed.


International Ry. Co. Power House
Loose rock and boulders to be removed from this area
Terrapin Point
STATE OF NEW YORK
GOAT ISLAND
Excavation in these areas to be determined after cofferdam is in place
Excavation in this area to be determined after cofferdam is in place
Shoal to be removed
PROVINCE OF ONTARIO
Excavation in this area to be determined after cofferdam is in place
Wing wall to be replaced by submerged masonry weir
Submerged masonry weir to be constructed if found necessary
Submerged masonry weir
HORSESHOE FALLS
Cofferdam
Cofferdam
INTERNATIONAL BOUNDARY
NIAGARA RIVER
Submerged masonry weir
Area between lines represents a flow of 10,000 sec.-ft. when the total river flow is 210,000 sec.-ft.
Canadian Niagara Power Company
0 100 200 300 400 500
Scale in Feet

DETAILS OF WORK TO BE UNDERTAKEN AT FALLS

The board submitted a further report to the Canadian and United States governments in May, 1928, in which it recommended that the joint proposal of the Ontario Hydro Electric Power Commission and of the Niagara Falls Power Company be accepted subject to conditions similar to those specified in the above proposals made by the commission and the company except that section 7 provides that "if, upon completion of said remedial works, the withdrawal of the additional 20,000 sec.-ft. or some part thereof shall not, in the opinion of the board, appreciably affect the scenic value of the falls and the integrity of the river, it is understood that diversion for observation purposes, referred to under section 5, may be continued only so long, not exceeding seven years from the date of beginning field construction, as may be necessary to enable negotiations to be undertaken and concluded for the modification of the present international treaty so as to permit permanent additional diversions of such amounts as may then be agreed upon."

According to the boundary water treaty between the United States and Great Britain concluded in January, 1909, the diversion within the State of New York of the waters of the Niagara River above the falls for power purposes, not exceeding 20,000 sec.-ft. in the aggregate per day, is permissible, and on the Canadian side 36,000 sec.-ft.

California State Parks Urged

Parks that would preserve California's various natural attractions are urged in a State Park Commission report recently prepared for the legislature. A $6,000,000 bond issue for park development, noted in *Engineering News-Record*, Feb. 9, 1928, p. 256, was approved last November and is to be spent when matched dollar for dollar by private funds. The report recommends 125 projects.

Progress of Tests and Designs of Bonnet Carré Spillway

Model Dam Tests Made to Determine Stilling Basin Dimensions to Dissipate Velocities—Upper and Lower Gates Indicated

A SPILLWAY above New Orleans to skim 250,000 sec.-ft. of the flood crest of the Mississippi River and discharge the water by a floodway into Lake Pontchartrain is one of the auxiliaries in the flood-control plan looking to the safety of New Orleans. Field tests on type of structure and foundation and designs have been progressing rapidly. The following notes were prepared by Major E. S. J. Irvine, in charge of the Bonnet Carré section, New Orleans River District, of which Major W. H. Holcombe, Corps of Engineers, U. S. Army, is district engineer. —EDITOR.

MODEL dam tests have reached the stage where it is possible to make some prediction as to the type of spillway structure best suited to the conditions at Bonnet Carré. Our experiments indicate that, for the depth of water flowing over the crest and for the height of fall, a stilling basin approximately 3 ft. in depth and 45 to 50 ft. in length will be required. These dimensions correspond very well with results determined in some of the German hydraulic laboratories. Baffle piers are not suited to dissipation of velocities under conditions at Bonnet Carré. In addition to the tests to determine the type of section of the weir, we also have channels below the flume laid out to study the critical scouring velocities and the critical silting velocities.

There is to be constructed an experimental table on which will be laid out a long model section of the spillway in order to study the lines of flow of the water approaching the spillway from the river and passing through the structure under various arrangements of gate openings. Also, on this same table, it is planned to set up a small-scale model of the floodway and study the hydraulics of the channel with various quantities of water running through it.

The hydraulic conditions at the spillway structure existing during high floods may be divided into two general classes: First, during periods of discharge of 50,000 to 200,000 sec.-ft.; second, for discharges greater than 250,000 sec.-ft. or for full capacity of spillway. Under both conditions the water level in the river will be close to 24 ft. mean gulf level. (This corresponds with a 20-ft. gage reading at Carrollton, which the project provides shall be maintained by diversions at Bonnet Carré.) Under the first condition it will be necessary to drop the water down to an elevation near natural ground level (12 ft. M.G.L.), the water running off the steeper slopes near the riverbank at practically free flow; in order to keep the velocities below the critical scouring velocities for the natural ground, it will be necessary to dissipate all possible energy at the spillway structure. Under the second condition, the tailwater will build up to considerable depths and it may become necessary to utilize all the available head to force the water through the floodway. With deep tailwater there is little danger of scouring action.

The logical design to cover both conditions with equal efficiency would be one providing for split gates—that is, the top gate to be pulled out for the initial discharge, the lower gate or gates to be raised after the backwater has been built up and a greater discharge desired. Experiments are now being conducted in the model dam flume simulating the flow of water with the upper gate raised, and raising the lower gates to observe the hydraulic effect. The greatest increase in velocity, of course, comes when the first of the lower gates is raised. This condition can be ameliorated by having the lower section covered by two gates, each of shallow depth.

It appears advisable to have the reserve head or capacity made available by these lower gates in event of a change in discharge capacity of the floodway, due to silting up; of failure to clear properly, thus forcing back deep tailwater on the dam; or in an emergency requiring a greater discharge than 250,000 sec.-ft.

It is believed that the use of movable gates instead of an ogee wall for the lower section of the dam would increase the efficiency of the stilling basin. This point is yet to be determined by experiments.

Studies have been made covering the following types of gates: tainter gates, wooden shutter gates, steel shutter gates with rollers, and Stoney roller gates. The present indications are that steel shutter gates with rollers will best fit conditions at Bonnet Carré.

Foundation Tests—Sixteen test holes, 50 to 100 ft. in depth, have been sunk along the spillway site. The soil strata vary at different locations, but the test holes showed that there is a uniform stratum of impervious blue clay and fine sand underlying the entire length of the foundation site. This stratum is 30 to 50 ft. in depth and generally lies 10 to 15 ft. below the ground surface. Our permeability tests indicate that this soil is practically impervious. Some of the tests showed no flow whatever with a pressure of 120 lb. per sq.in. on the samples. This stratum is very favorable to the prevention of percolation under the dam. With a deep cut-off wall at the upstream face of the spillway and a shallower one at the downstream end, each of steel sheetpiling, the dam will be absolutely safe against cutting through.

Other strata cut through included variations of sand, humus and blue clay. At one point quicksand was encountered at a depth of 60 ft. Several holes were bored around this locality to insure that it did not extend up through the stratum of impervious clay.

Fifteen wooden and ten concrete piles have been driven along the spillway site. The *Engineering News* formulas indicate that the wooden piles should bear between 14 and 32 tons and the concrete piles between 18 and 36 tons. However, the piles loaded to date indicate that these formulas cannot be depended upon for the type of foundation soil in which we are working. One wooden pile has been loaded to 25 tons without failure, and a concrete pile has been loaded to 45 tons with a settlement of approximately 0.18 ft.

The tests to determine the bearing value of the soil are giving good results. We have loaded four posts, each 1 sq.ft. in area, up to 2,600 lb. per sq.ft. without large settlement. We are continuing loading slabs of varying dimensions. Upon the completion of these tests the bearing value of the surface for large areas will be computed according to Terzaghi's formula.

It is hoped to have the detailed designs ready to submit to the Chief of Engineers early in March. Work will start on the construction for the lower levee of the floodway as soon as right-of-way are acquired. The schedule now being worked on calls for completion of the entire project by January, 1931.

Letters to the Editor

A Forum for Discussion of Views of Engineers and Contractors

Horizontal Forces on Bascule Bridges

Sir—In reading your editorial comment and the general article on the failure of the counterweight tower of the bascule bridge over the Hackensack River (*Engineering News-Record*, Jan. 10, 1929, p. 74), it occurs to me that it might be of interest to note the clause in our Canadian Engineering Standards Association Specification for Movable Bridges, A-20-1927, where such supports are dealt with. Clause 31 provides that "steel towers and special parts of the structure which support the counterweight for bascule and vertical lift bridges shall be proportioned to resist, in addition to the vertical loads, a horizontal force in any direction equal to

(*a*) Wind load resulting from 30 lb. per sq.ft. of the total exposed area of the structure as projected onto the vertical plane normal to the wind; except that one truss shall not be considered as sheltering the other whatever may be the direction of the wind; or

(*b*) 5 per cent of the supported load applied at its center of gravity, whichever demands the greater section."

Montreal, Que.
Jan. 25, 1929.

P. L. PRATLEY,
Consulting Engineer.

Eccentricity Assumptions in Bascule Bridge Design

Sir—The wreck of the Hackensack bascule bridge, as you rightly point out in your recent editorial, is a matter that should be subject to searching and satisfying scrutiny. Any light that can be thrown on it should be welcome. The investigating board's report, as published in your issue of Jan. 10, 1929, p. 74, undoubtedly points out the source of the weakness and correctly describes the sequence of events.

I think, however, that by postulating a few natural assumptions the thing can be reduced to one of calculation and taken out of the mysterious or even judgment class.

The 770-ton counterweight was manifestly intended to be made with its center of gravity exactly in the center plane of the two posts on which it rested. It would be a very difficult matter to construct this so accurately that the center of gravity would be exactly in this plane. Assuming that it was, say, 3 in. out of the intended position, there would be a couple of 4,620,000 in.-lb. to provide for. This would mean a force at the top of the counterweight towers of about 11,000 lb., or 5,500 lb. in each tower. These towers were lattice posts with slender lattice and were doubtless not designed for a bending moment of 2,300,000 in.-lb. in each post and 5,500 lb. of shear.

Furthermore, this assumed eccentricity of load on the posts supporting the counterweight would account for a greatly increased stress in those posts, both direct stress and shear. It could readily account for the incipient failure observed in the post of the west structure. Since the eccentricity in the counterweight might even be more than 3 in., it would be the part of wisdom to assume it to be about 6 in. if not more and to design the parts accordingly.

Pittsburgh, Pa.,
Jan. 16, 1929.

EDWARD GODFREY,
Consulting Engineer.

Aggregate Moisture Determination by Specific Gravity

Sir—In your issue of Dec. 27, 1928, p. 966, H. C. McCall raises several seemingly just criticisms of the specific gravity principle of determining moisture content.

For the information of Mr. McCall and others who are interested in this test the following observations are made in reply to his letter. Since the brief description of the method in the issue of Nov. 15, 1928, study has been made in our laboratory of the very factors mentioned by Mr. McCall. On sands tested in the Iowa State College laboratory none was found to have a total percentage of absorption of more than 0.6 per cent. Obviously an error of 0.5 per cent of the weight of the sample in determining the "saturated and surface dry" condition would be very difficult to make from the standpoint of getting the sample too dry. However, even if this error should be present in determining the saturated condition of the sand, it is still present if moisture is determined by the flame drying method or by any method so long as absorption is considered at all. Specific gravity tests from weight displacement on completely dry and on saturated and surface dry sands show that the weight of a dry sand and a saturated and surface dry sand are nearly identical at the end of ten minutes. However, the dry sand, if allowed to remain in water for 24 hours and freed of trapped air, will show an increase in weight due to the liberation of the air.

Variations in the specific gravity of sands from the same source are rare. Of the local sands at Ames none has been found to vary enough to influence the test. Sands in general vary from one another from 2.63 to 2.67, too great a range to allow the use of general constants in moisture determination.

Since the specific gravity determines the quantity of material in the mix when proportioning on an absolute volume basis, any variation should be known. The method of determining the moisture content will not eliminate this factor; it is present in the flame drying method or in the specific gravity method.

Absorptive stone soaked for twenty minutes in the apparatus introduces an error up to 50 per cent if the equations published are used as shown. These equations are not correct for this test until W'_1 has reached the same state of saturation as that at which W'_2 or G_s was determined. For example, a sample has a W'_2 at twenty minutes equal to 1,235; if allowed to remain for 24 hours, its absorption will increase 20 per cent, or W'_2 to 1,242; W'_1 equals 1,221, or G_s equals 2.57. For 1,235, x equals 24 grams; for 1,242, x equals 35 grams, the correct value. Study of the rate of absorption of such aggregates has enabled the author to present a quick and practical method for eliminating this error.

Students in the materials classes have checked both free moisture and absorption by the specific gravity principle against flame drying for over a year. With the type of apparatus now in use a variation of more than 0.1 per cent in moisture content is rare and 90 per cent of the tests are within that. Such accuracy has entirely convinced us of the practicability of this test in concrete work. It is well within the accuracy of the relations establishing the water-cement ratio curves.

Ames, Iowa,
Jan. 22, 1929.

W. M. DUNAGAN,
Instructor of Materials,
Iowa State College.

Engineers Entertain Legislators

Sir—I am inclosing a program of a dinner recently given by the engineers of St. Louis to the members of the Missouri Legislature from St. Louis and St. Louis County. This was the third biennial dinner given by the engineers to the legislators for the purpose of creating a clearer understanding of the province of the engineer in the minds of the legislators, and also to promote a more cordial feeling between them as they became better acquainted with each other.

The engineers sought to impress the legislators with the fact that they had no interest in any particular legislation which was likely to come before the legislature, but offered to advise them on any legislation that came up which was related in any way to engineering.

The engineers here feel that these dinners have been worth while, and that, in time, they are bound to have a good effect, especially on all legislation in city and state affairs in which engineers are directly or indirectly interested. I thought that this would be interesting to you and to others interested in developing a better understanding of the province of engineering.

St. Louis, Mo.,
Dec. 31, 1928.

EDWARD E. WALL,
Consulting Engineer.

The program listed but three formal speeches: Legislation and City Planning, by Harland Bartholomew; Dams, by Robert E. McDonnell; County and District Improvements, by Albert P. Greensfelder. Twenty-one legislators were guests.—EDITOR.

News of the Week

Illinois River Flood-Control Program Before Legislature

Flood-control work on the Illinois River to cost $3,000,000 will be recommended by the Illinois Valley Flood-Control Association in a report to the Illinois General Assembly, in session at Springfield.

The five methods of flood prevention urged by the association are: storage basins at headwaters of tributary streams to defer flow of water at flood seasons; relocation of levees to widen the river basin; utilization of some existing levees and the addition of others as storage facilities in times of flood; deepening and widening the stream; and reducing the diversion of water from Lake Michigan into the Illinois River.

The report also urges co-operation between the state and the federal agencies relating to river matters. Widening of the river between Beardstown and Pearl, a distance of 40 miles, is recommended.

1929 Construction by A.T.&T. Will Total $82,000,000

Long-distance telephone extensions worth $82,000,000 will be installed this year by the American Telephone & Telegraph Company. This is 67 per cent more than the amount spent for similar work in 1928, which was a record. About half the expenditures will be for extending and supplementing the existing long-distance cable systems. The extension will total 2,500 miles. In less populous areas 74,000 miles of wire will be strung on new pole lines at a cost of $10,000,000.

Chicago Sanitary Board Engages Eleven Consultants

Appointment of eleven widely known sanitary engineers by the Sanitary District of Chicago has been made to make a survey of the present engineering situation and to define limits of the program required in light of the recent adverse Supreme Court decision against the use of lake water for sewage dilution proposes. The engineers held their first meeting Feb. 11 to receive instructions from E. J. Kelly, chief engineer of the district. The following engineers have been retained: Harrison P. Eddy, George W. Fuller, John H. Gregory, E. E. Haskell, Asa Phillips, Ezra B. Whitman and Richard R. Lyman, consulting engineers; George H. Fenkell, superintendent and general manager, Detroit Water Supply; W. W. De Berard, associate editor, *Engineering News-Record;* Major General Harry Taylor, U. S. Army (retired), and Ansen Marston, dean of engineering, Iowa State College.

New York State Gets Its Rights Protected in the Niagara Treaty

State Control Over Water Diverted in Remedial Work Studies Fully Recognized

FOLLOWING the presentation to the United States Senate of the treaty between the United States and Canada, recently signed at Ottawa, for the construction of weirs and other works above Niagara Falls (abstracted on p. 281 of this issue), Senator Robert F. Wagner of New York raised the question in a letter to Governor Roosevelt as to whether the rights of New York State were fully protected under the proposed treaty. The treaty provides for the diversion of 10,000 sec.-ft. on each side of the river in winter during a period of seven years following the construction of the remedial works above the falls in order to study the effect of the diversion upon the falls. This water is to be allotted to the Niagara Falls Power Company on the American side and to the Hydro Electric Power Commision on the Ontario side, and no reference is made in the treaty to New York State's rights in this water or its control over the land under water in the river upon which the remedial works are to be built.

In reply to Senator Wagner, Governor Roosevelt states that New York State was not asked to participate in the negotiations preceding the treaty despite its interest both in the falls itself and in the water diverted for power purposes. He adds, however, that the State of New York will offer no objection to the ratification of the treaty, because it is deeply interested in the preservation of the scenic beauty of Niagara Falls and believes that its rights in the matter can be fully protected. Moreover, since the signing of the treaty, the Governor has, through direct negotiations with the Niagara Falls Power Company, obtained its signature to an agreement that if it is allowed to use for seven years the additional water obtained from the experimental diversion, it will withdraw all objections to the payment of the rental now being paid to the state for existing waters under protest, will agree to pay such equitable rental for the additional experimental supply as may be determined by the state under the water-power act and will obtain a license from the state for use of the experimental water. This, states Governor Roosevelt, is a complete recognition of the rights and sovereignty of the State of New York to license and to rent all water now being used and all water to be used under the new treaty.

Detroit Water-Supply Extensions Upheld by Engineers

Detroit's project, now under way, for increasing its present water supply from the Detroit River rather than go to Lake Huron has been approved by Prof. William C. Hoad, Clarence W. Hubbell and Gardner S. Williams, consulting engineers to whom the matter was referred for opinion. A supply from Lake Huron, it is estimated, would cost from $35,000,000 to $60,000,000 more than the additional supply now being provided and the water would be no better, the engineers report. A supply from Lake Huron would have to be pumped 60 miles.

Moffat Cutoff Hearing Feb. 26

Oral argument on the application of the Denver & Salt Lake Western Railroad for authority to build a 42-mile cutoff connecting its line and the Moffat railroad in Colorado will be heard by the Interstate Commerce Commision on Feb. 26.

Illinois Again Proposes Gas Tax

A bill providing for a gasoline tax of 3c. a gallon, estimated to add nearly $27,000,000 annually to the roadbuilding fund of the state, has been introduced into the Illinois Legislature at the recommendation of Governor Emmerson. The bill is drafted to overcome objections found in the recent gasoline tax law invalidated by the state Supreme Court as unconstitutional, but carries the same provisions for exemptions as the former law.

Canada Halts Discussion of St. Lawrence Waterway

As the speech from the throne on the opening of the Dominion Parliament Feb. 7 omitted all reference to the St. Lawrence waterways improvement project, it indicates that no action will be taken during the present year for the renewal of negotiations with Washington on the question. The dispute as to whether the water power to be created by the work belongs to the dominion or to the provincial governments of Ontario or Quebec, which was the cause of the breaking off of the negotiations, is still unsettled. The question was referred to the Canadian Supreme Court, but its decision was inconclusive, the judges declaring that the questions submitted to them bearing on the matter could not be answered categorically. Until this matter is decided either by a definite legal judgment or by agreement between the dominion government and the provinces concerned, the whole scheme will remain in abeyance.

Chicago Regional Body Hears of Highways and World's Fair

Regional highway development, rights-of-way, zoning, park and forest preserves and a warning to realtors against using the proposed World's Fair as selling bait were topics discussed at the sixth annual meeting of the Chicago Regional Planning Association on Feb. 5. Daniel H. Burnham was reelected president and Robert Kingery general manager and secretary. A budget of $85,000 has been laid out for 1928.

H. R. Olsen, highway engineer of the association, traced the history and development of the superhighway, defining it as a highway 160 to 200 ft. wide, including grade separation and facilities for high-speed traffic on a different lane from that carrying the slower and heavier traffic. Especial attention was called to the decided tendency in the Chicago region for rapid-traffic rail service to seek private rights-of-way, materially simplifying the design, since in that case only vehicular traffic needs consideration.

Hugh Young, chief engineer of the Chicago Plan Commission, analyzed the elevated highway. Estimated costs of various kinds, exclusive of the factor of unknown damage to adjacent property, ranged from $2,000,000 up per mile. He did not favor narrow elevated highways built on an ordinary 66-ft. right-of-way.

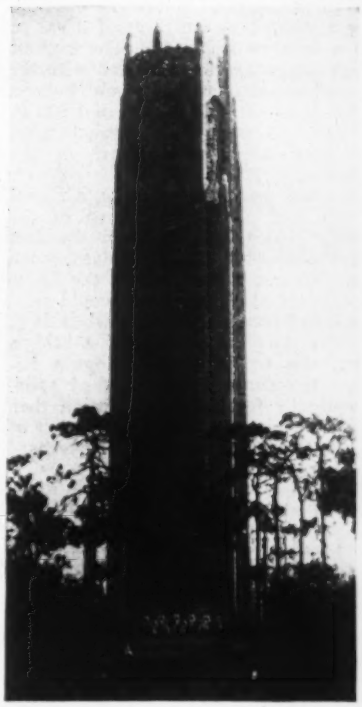
L. C. Holleran, deputy chief engineer, Westchester County (N.Y.) Park Commission, spoke of six years' work in cleaning up and parking the streams of that county, buying 16,000 acres of land and building 140 miles of parkways.

"Street Widening by Setback Lines," by Judge C. E. Randall, Kenosha, Wis., was the subject of a paper of legal value to authorities confronted with the problem. The acquisition of additional right-of-way can be accomplished by original plat dedication, by voluntary action, by condemnation, by legislation, by using the right of eminent domain and by the use of police power, according to Judge Randall.

The grade separation program of Illinois was outlined by H. E. Surman, engineer of design of the state Highway Department. The program is for 200 in the next ten years. Downstate separations cost $75,000 each, but in the Chicago region the cost is $185,000. There are 50 of the latter on the state bond issue system.

Allen D. Albert, assistant to the president of the World's Fair, told the 400 members and guests at the evening session that no extra labor and no more subdivision of land are needed. People coming to the fair will not settle in Chicago and only the normal population increase of 120,000 annually is to be provided for. Self-restraint on the part of the realtor was urged to avoid speculative expansion of Florida-boom proportions—and results. Studies indicate that the extra 270,000 lots platted in the 1893 fair year were enough to last 27 years.

UNUSUAL CARILLON TOWER DEDICATED IN FLORIDA



A 205-ft. carillon tower of unusual design was dedicated Feb. 1 by President Coolidge at Mountain Lake, Fla. The tower was built as part of the plan of Edward W. Bok to create "for the American people the most beautiful spot of verdure in the United States." The base of the tower is 51 ft. square and at the 150-ft. level the tower changes to an octagonal plan, measuring 37 ft. at the top. The structure consists of a structural steel frame inclosed in brick walls faced with Georgia marble and tan Florida coquina rock. The upper portion of the tower contains 61 bells, the largest of which weighs 11 tons and the smallest about 17 lb.

Large Foreign Contracts Proposed

Several large foreign contracts are under way or proposed, according to the U. S. Department of Commerce. A contract is pending for a concrete pier at Cebu, P. I., which will be 505 ft. long and 107 ft. wide supported on 615 concrete piles. The project also includes a marginal wharf resting on 338 piles. The estimated cost of the project is $556,000 and it is to be completed within 750 days.

A concrete wharf from 1,900 ft. to 2,600 ft. long is planned at Lobito, on the west coast of Africa. Construction of this project is contingent upon raising a loan by Portugal.

At Ghost and Bow rivers 35 miles west of Calgary, Alta., a contract has been let for a power dam more than half a mile long.

High Level Viaduct Planned for Ninth St., Richmond

Surveys are being made for a new high level highway bridge across James River at Richmond, Va., replacing the present Ninth St. structure, which has served for three decades. In 1913 there were 32 plans submitted for a bridge at this site in a competitive design campaign. The winning design, similar to that contemplated at present, was submitted by B. H. Davis, of New York, now deceased. The J. E. Greiner Company, of Baltimore, has been retained as consulting engineer on the proposed structure, which is estimated to cost $750,000.

$175,000,000 of Highway Bonds Recommended for Texas

The Governor of Texas has recommended to the legislature that highway bonds to the amount of $175,000,000 be authorized. It is forecast that the recommendation will pass both houses. The election to pass on the authorization is expected to be held in the fall. If the vote is favorable, the necessary enabling act would come before the legislature in 1931. To take care of the situation meanwhile the Governor's committee of seven recommends that for the next two years the Highway Commission be supplied from state funds with $20,000,000 annually for construction and $10,000,000 for maintenance, this income to be derived from a tax on gasoline and registration fees as the legislature may by law provide.

Samuel E. Emmons to Manage Railway Properties in Brazil

Samuel E. Emmons, a civil engineer who has since April, 1924, been assistant general manager of the United Railways & Electric Company, Baltimore, has resigned, effective Feb. 1, to become associated with the American Foreign Power Company, controlled by the Electric Bond & Share Company. He will leave on Feb. 9 for Rio de Janeiro and take up the management of railway properties owned and controlled by the American Foreign Power Company in Brazil.

Mr. Emmons received his degree in civil engineering from the University of Michigan in 1916 and took up work with the Atchison, Topeka & Santa Fe Railroad. He went with the Pennsylvania Railroad later, his service with that company being interrupted while he served as a lieutenant and captain in the Fifteenth Regiment of Railway Engineers during the war. Resigning from the Pennsylvania, Mr. Emmons became associated with Professors Cooley, Riggs and Anderson on valuation work. He became assistant chief engineer of the Ann Arbor Railroad in 1920, leaving in September, 1922, to become assistant to the general manager of the United Railways & Electric Company, Baltimore. He was promoted to assistant general manager in April, 1924.

Regional Planning Proposed in Hamilton County, Ohio

Initial steps toward the creation of a regional planning commission for Hamilton County, Ohio, were taken on Feb. 6 at a meeting attended by official representatives of the county, the city of Cincinnati and other municipalties within the county. A second meeting is tentatively scheduled for March 6, with County Commissioner Charles Urban as chairman the representatives of planning commissions in the county in attendance.

WASHINGTON NOTES

By Paul Wooton
Washington Correspondent

Private Development of Colorado River Threatened—New Bridge Over Delaware River Meets Opposition

A THREAT to open the Colorado River for private power development has been raised by Boulder dam proponents, who believe that in this manner they can force the approval of the states now opposed to the project. The present act restricting private power development on the Colorado expires on March 4 and the interstate commerce committee of the House soon will take up a resolution to extend the restriction.

Indications were given by Representative Lea, of California, during preliminary discussions of the subject that he will oppose action to extend the limitation. Representative Swing also declared that apprehension will be instilled into the reluctant states if they believe there is likelihood of private enterprise acquiring prior power rights in the vicinity of Boulder Canyon.

Advancing still another thought on the subject, members of the House committee on reclamation have suggested that the Boulder dam advocates may really desire to leave the way open for private interests to develop the river should the present project become ineffective through the failure of either Arizona or Utah to ratify the compact.

Delaware River Bridge

The War and Navy departments are vigorously opposing the erection of a bridge over the Delaware River below Philadelphia, it was brought out at the hearings before the interstate commerce committee of the House on the proposed Wilmington bridge. Representatives of the Philadelphia harbor district and maritime and shipping interests also testified against the project. Although the committee itself has taken no action on the measure authorizing the construction of the bridge, the Pennsylvania delegation in the House has adopted a resolution strongly opposing the project.

Major Gen. Edgar Jadwin, Chief of Engineers, conceded that a bridge could be built across the Delaware with clearances sufficient to allow the largest ships to pass. Replying to questions put by members of the committee, he said that a bridge to provide ample clearances for present-day craft should have a width between piers of 2,500 ft. and a height of 200 ft. The highest bridge suggested in connection with the bill under consideration would have a height of 173 ft. and a span of 1,800 ft. He said no one could predict what clearances might be needed in the future.

Of even more consequence, General Jadwin said, is the vast area of the country which benefits from the fact that Philadelphia is a first-class ocean port. To put a bridge over the Delaware River at Wilmington would relegate it to a secondary status, when there is really no good reason for a bridge. When the traffic at Wilmington becomes too dense to be handled satisfactorily by ferries, consideration then should be given to the construction of a tunnel rather than an overhead crossing.

Appearing in behalf of the navy, Rear Admiral Gregory, chief of the bureau of yards and docks, stated that any bridge across the Delaware below Philadelphia would be detrimental to the operation of the Philadelphia Navy Yard, in which the government has a capital investment of $195,000,000. Although of the opinion that no bridge could be built at the location under consideration without interfering seriously with naval operations, he raised particular objection to the clearances intended for the proposed bridge. Many of the present naval craft would be unable to pass under a bridge with a vertical clearance of 173 ft., he said.

Hydraulic Laboratory Bill

Representative Wallace Dempsey, of New York, chairman of the rivers and harbors committee of the House, with the active support of two or three members of the committee, is being accused by proponents of the national hydraulic laboratory bill of employing dilatory tactics in his effort to prevent the passage of the measure. The bill already has passed the Senate. After having held the bill in his committee for nearly ten months, Chairman Dempsey on Feb. 9 suggested a compromise under which the bill would be made a part of the next rivers and harbors bill. If the bill fails at this session, an effort will be made to have it referred in the next Congress to the flood-control committee. Many believe that the reference of this bill to the rivers and harbors committee was an error.

Muscle Shoals Bill

With the legislative way paved for the passage of another Muscle Shoals bill at this session, the opposition scored an unexpected victory in preventing, with a margin of one vote, the reporting out of the bill by the military affairs committee of the House. As this is written, an effort is being made to work out a compromise, but precious time has been lost, with the indications pointing to no further legislation at this session. The proposal under consideration is that of the American Cyanamid Company, but as it calls for the construction of Dam No. 3 and of the Cove Creek dam with federal funds, the plan in its present form conflicts with the President's determination that the government incur no further expense in connection with the project.

Safeguard for Great Lakes

An attempt to reduce to a minimum flow any diversion of water from the Great Lakes that the advocates of the lakes-to-the-gulf waterway project may seek to obtain is seen in a bill recently brought up in the House by Representative Peavey, of Wisconsin. After declaring that as a matter of policy the waters of the Great Lakes should be retained within their natural watershed, the bill proposes to make an exception so as to permit diversion to adjoining watersheds to aid navigation during times of low flow. It proposes further that the Secretary of War may issue revocable permits for the diversion of as much water as the Chief of Engineers believes to be necessary. Before any permit can be issued, however, the bill makes the provision that it must be proved that every other engineering and economic means has been resorted to for the furnishing of enough water for navigation.

Federal Aid for Drainage Districts

Hearings were completed recently by the House irrigation and reclamation committee on the Smith bill authorizing an appropriation of $180,000,000 to aid drainage and levee districts to redeem their outstanding bonds. Under the terms of the bill the appropriation is to be used by the Secretary of the Interior as a revolving fund for the purpose of making non-interest-bearing loans to drainage districts for periods not to exceed 40 years. In order to provide security, it is specified in the bill that the drainage district applying for aid must deposit with the Interior Department non-interest-bearing refunding bonds to an amount equal to the loan sought.

As the result of the hearings, it was brought out that sections of 34 states, aggregating 65,634,743 acres of land in drainage, would be benefited by the bill. It was developed further that the amount invested in these drainage enterprises is $390,040,043 and that there is a farm population totaling 5,000,000 living in the area affected. The drainage taxes on much of this acreage equal all other taxation, and there is urgent need of relief, it was claimed.

Revision of Testing Laboratories List

The growing demand for other laboratories than that of the Bureau of Standards in which commodities may be tested by the specifications has led the bureau to plan a revision of its list of such laboratories, and a questionnaire has been sent out asking for data to be included, such as a complete list of testing machines, whether commercial tests are made and the kind of work preferred.

Consolidation of Cleveland and Cuyahoga County

A plan designed primarily for the consolidation of the city of Cleveland with Cuyahoga County but applicable to the seven other counties of 150,000 population or more in Ohio has been formulated and a constitutional amendment authorizing it has been drawn for introduction in the legislature. The plan is the work of the "fact-finding and policy committee of the regional board of 400," according to the Cleveland *Plain Dealer* of Jan. 29. William A. Greenlund, former Lieutenant Governor of Ohio, is chairman and Leyton E. Carter is secretary of the committee. Under the plan, the four cities, 49 villages, 6 townships and 33 school districts in the county would be left undisturbed in purely local affairs, but the way is prepared for unified control of highways, water supply, sewage disposal, street cars and rapid transit, public health and parks in so far as these are or might become county wide.

Legal Attack on Iowa Highway Bonds

Terming it "good financing but an open, palpable violation of the state Constitution," the Attorney General of Iowa is opposing in the state Supreme Court the $100,000,000 bond issues for paving. The fate of the bill providing these funds rests on whether the law conforms to the Iowa Constitution, requiring the imposition of a direct tax, and whether it distinctly states the tax. The Constitution provides that bond issues shall be retired by a direct general tax. The bill provides that primary road funds, indirectly collected, shall be used to retire the bonds by supplementing the direct tax. The contention is that the amount of the general tax is not definitely stated.

Brief News

A PRIVATE HIGHWAY TOLL BRIDGE across Puget Sound at the Narrows in Tacoma to be built at a cost between $7,000,000 and $10,000,000 is proposed. The bridge would be the first to be built across the sound.

THE DEPARTMENT OF CIVIL and Sanitary Engineering of the Massachusetts Institute of Technology, Cambridge, Mass., will hold a two-day conference for the Massachusetts State Association of Master Plumbers and Massachusetts Sanitary Club Feb. 19 and 20.

A $10,000,000 HIGHWAY BOND ISSUE will be sold to the highest bidder on Feb. 26 by the State of Louisiana. The bonds will constitute a direct and general obligation of the State of Louisiana and not a limited tax issue, according to attorneys retained by the state to decide the status of the issue. Interest on the bonds is limited to 5 per cent, and the loan will be repayable principally from tax on motor fuel.

A DEFINITE PROGRAM of tree planting along Tennessee's highways will be placed in charge of division engineers of the state highway department.

TEST DRILL HOLES are being made on the site of the proposed Rock Island Rapids dam in the Columbia River 10 miles from Wenatchee, Wash. One test hole has reached a depth of 117 ft.

TRAFFIC CONTROL DEVICES costing $1,000,000 are recommended for Philadelphia by Superintendent of Police Mills, who declares that their installation would reduce traffic accidents in the city 50 to 75 per cent.

A PENALTY of 6 mills per capita per day for each offense would be imposed on any public sewer system discharging sewage into waters of the state without a permit, according to a proposed amendment to the public health law introduced into the New York State Senate.

EXCAVATION HAS COMMENCED for the Stewart Mountain dam in Arizona. The dam will be the fourth in the chain of the Salt River Valley Water Users' Association. C. C. Cragin, chief engineer of the association, is in charge of construction, with J. S. Connell as assistant.

A NATIONALLY KNOWN underground water geologist would be required to determine the source of all underground municipal water supplies in the State of Texas, according to terms of a bill prepared for the state legislature.

THE 2-MILE HIGHWAY BRIDGE across Sandusky Bay near Sandusky, Ohio, was opened to traffic after dedication ceremonies on Feb. 2. By providing a direct route from Sandusky to Port Clinton, the bridge shortens the route between Cleveland and Detroit by 25 miles. The structure cost $2,000,000 and will be operated as a private toll project.

THE 29-STORY STERRICK BUILDING under construction at Memphis, Tenn., will be completed about Sept. 15. The structure will be the tallest in the city.

THE NATIONAL ASSOCIATION OF BUILDING TRADES EMPLOYERS at the annual construction conference in Chicago, Jan. 25, passed a resolution demanding that the building trades department of the American Federation of Labor and other agencies study the problem of the five-day week and find its solution or form a commission for this purpose.

WORK HAS COMMENCED on the construction of the Hamilton Northwestern railroad, 10 miles of new line running north from Hamlin, Tex.

THE LEGAL WIDTH of mountain roads in Colorado will be determined by a suit filed in the district court at Golden recently. The county highway department claims that under the grant of 1883 it has a right to a width of 60 ft. and is taking advantage of that right. The landowners claim that the old wagon-road width of 18 ft. is all the county is entitled to.

Personal Notes

BENJAMIN HINKEY has been appointed city engineer of Tonawanda, N. Y., succeeding THOMAS W. BARRALLY. Mr. Hinkey formerly held the position, resigning in 1917.

E. S. FICKES, vice-president in charge of engineering of the Aluminum Company of America, has been elected a director of the company.

A. D. BUTLER, city engineer of Spokane, Wash., has been elected president of the Associated Engineers of that city.

EWING TOWLES, assistant chief engineer of the Missouri State Public Service Commission, has resigned effective March 1.

CHARLES H. WILLIAMS has been appointed city engineer of Olympia, Wash., to succeed CHARLES I. SINGER, resigned. Mr. Williams was for nine years assistant district engineer with the Washington State Highway Department, with headquarters at Chehalis.

LARRY O. COX has been appointed division engineer for the eighth division of the Texas State Highway Department, succeeding J. B. EARLY, who has been appointed state maintenance engineer. Mr. Cox has been, for the past three years, resident engineer for the Texas Highway Department in Carson and Hale counties. He will have division headquarters at Abilene, Tex.

W. K. MYERS has been appointed assistant maintenance superintendent for the Pennsylvania Department of Highways in Monroe and Pike counties, with headquarters in East Stroudsburg. He was formerly engineer and superintendent of operations for the Mapleton Limestone Company, Huntingdon, Pa., and before that was maintenance superintendent for the Pennsylvania Department of Highways in Cambria County.

FRANK V. TAIT, recently employed by the Port of New York Authority, is now assistant engineer on the New Jersey section of the Hudson River bridge under Frank E. Cudworth, resident engineer. Previously he was empoyed by the Pennsylvania Highway Department as assistant engineer in charge of township roads.

GAIL A. HATHAWAY, recently employed by the office of the Oregon state engineer, in charge of the department of water distribution, hydrographic surveys and snow surveys, has accepted a position as hydraulic engineer with the Kansas City District office of the U. S. Army Corps of Engineers. His present work is in connection with the investigations of the Missouri River for flood control, navigation, irrigation and power.

Engineering Societies

CALENDAR

ANNUAL MEETINGS

ASSOCIATED GENERAL CONTRACTORS OF AMERICA, Washington, D. C.; annual convention, Chicago, Ill., Feb. 18-22.

NATIONAL DRAINAGE CONGRESS, Kansas City, Mo.; annual convention, Memphis, Tenn., Feb. 19-21.

AMERICAN RAILWAY ENGINEERING ASSOCIATION, Chicago; annual convention, Chicago, March 5-7.

AMERICAN SOCIETY OF CIVIL ENGINEERS, New York; spring meeting, Dallas, Tex., April 24-26.

* * *

THE AMERICAN WATER-WORKS ASSOCIATION, FLORIDA SECTION, will hold its annual convention in Jacksonville April 10 and 11. Ellsworth L. Filby, chief of the bureau of sanitation, Florida State Board of Health, is secretary of the section.

THE NORTHWEST SOCIETY OF HIGHWAY ENGINEERS elected the following officers at a meeting held Jan. 26 in Portland: L. P. Campbell, equipment engineer of the Oregon State Highway Commission, president; J. S. Sawyer, vice-president; C. G. Ryder, treasurer; A. A. Amort and E. A. Collier, directors.

THE SAN DIEGO (CALIF.) SECTION of the American Society of Civil Engineers has elected the following officers: President, Paul R. Watson, of Watson, Valle & Gough; vice-president, W. W. Crosby; secretary-treasurer, H. A. Noble, engineer with the San Diego Consolidated Gas & Electric Company.

CROSS-CONNECTIONS between approved public potable water supplies and other sources of supply will be discussed at the annual conference of state and local health officers, Trenton, N. J., Feb. 15. The subject will be considered in the light of Chap. 13 of the State Sanitary Code, reprinted in *Engineering News-Record*, Oct. 18, 1928, p. 602.

THE COLORADO SOCIETY OF ENGINEERS has appointed local representatives in all of the important cities of the state to keep the parent body posted on engineering projects and bring in new members of the association.

Obituary

T. BARTON JONES, 77 years of age, a civil engineer who was engaged in railroad work for many years, died at his home in Baltimore on Feb. 6. As a young man, while associated with the Baltimore & Ohio Railroad he assisted in laying out and constructing the Valley Railroad in the Shenandoah Valley. He was engineering representative of the late John W. Garrett when the latter was president of the Baltimore & Ohio. He also was closely associated with the late Samuel Spencer, who was vice-president and then president of the Baltimore & Ohio. When Mr. Spencer was made president of the Southern Railroad Mr. Jones was given contracts to double track more than 500 miles of the Southern and for constructing several long branch lines. When Mr. Jones became a member of the firm of Jones & Thorne he was in charge of many important Baltimore municipal sewer and bridge projects.

W. G. MEASELL, 26, engineer with the Norfolk & Western Railroad, was killed Feb. 5 in a cave-in of a tunnel under construction for the road near Devon, W. Va.

PHILIP SELLERS, New Haven architect and former president of the New Haven chapter of the American Association of Engineers, died in Philadelphia Feb. 6 from injuries received in an auto accident in that city.

GEORGE H. WETHERBEE, consulting engineer, of Boston, died Feb. 2 at his home in Braintree, Mass. Mr. Wetherbee graduated from the Massachusetts Institute of Technology in 1891.

MICHAEL L. QUINN, of New York, well-known tunnel builder, died on Jan. 21. Some of the contracts with which he was connected, usually as general superintendent, were: seven deep shafts and about 5 miles of tunnel in upper Manhattan for the Board of Water Supply, in 1911; the William St. river tunnels and the Battery tunnels to Montague St., Brooklyn, and the tunnel from Flatbush Ave. and Willoughby St. to Borough Hall, Brooklyn, in 1914; the Fourteenth St. river tunnels and open-cut section of subway from Irving Place to Sixth Ave., in 1917; the Liberty tunnels, Pittsburgh, in 1920; and the Manhattan side of the Holland tunnel, in 1922. During the past year he completed a tunnel in New Brunswick, Canada.

Cost and Contracts

E. N.-R. Index Numbers

Cost		Volume		
Feb. 1, 1929	**210.40**	**January, 1929**	**325**
Jan. 1, 1929	209.40	December, 1928	244
Feb. 1, 1928	204.65	January, 1928	208
Average, 1928	206.78	Average, 1928	287
Average, 1927	206.24	Average, 1927	263
1913	100.00	1913	100

This Week's Contracts

Heavy construction contracts, reported by *Engineering News-Record* in the week ended Feb. 11, with some comparisons, total as follows:

(In Thousands of Dollars)

	Feb. 11, 1929	Jan. 31, 1929	Feb. 14, 1928
Building:			
Industrial	$11,625	$8,339	$2,413
Commercial	70,646	17,446	20,812
Streets and roads	2,161	4,889	13,179
Other eng. constr.	4,597	21,328	9,297
Total	$89,029	$52,002	$45,701
Total, all classes, Jan. 1 to Feb. 11:			
1929			$633,089
1928			353,340

Random Lines

Niagara Power Plants to Be 142 Per Cent Efficient

The following despatch to the New York *Times* indicates that revolutionary changes are about to take place at Niagara Falls, to say nothing of all our cities where the new motors are to be attached to the water mains to light and heat the houses:

Asbury Park, N. J., Sept. 15 (A.P.).—A water motor which, its inventors say, would increase the output of the Niagara Falls power plant 50 per cent with the same volume of water has been perfected by three men of Monmouth County. The inventors are William T. Plum, Jr., of Deal; Albert M. Elder, of Elberon Park, and Sidney D. Van Dyke, of Wanamassa.

More than two years ago Plum conceived the idea of a double-action water motor which, by making ejection of water from the cylinder as speedy as injection, would, in the inventor's words, revolutionize development of water power.

When Plum got his idea he went to Van Dyke, a plumbing contractor, and together they sought knowledge of mechanical details from Elder, an engineer who had helped develop a high-pressure fire-fighting engine.

The inventors see a refinement of the motor to such a point where the power developed will light and heat homes through its attachment to street mains.

Good-by Transit!

The San Francisco *Chronicle*, in an editorial on aerial surveys, makes the following conservative statement:

But in Europe the attention given to aerial surveying has resulted in a great technical development of special cameras for the work and of extraordinarily clever machines for reducing air photographs to maps.

Along with air survey has developed in Europe a remarkable new process of survey from the ground, especially adapted to mountain regions. It uses a precision camera combined with a theodolite. This produces a pair of exact negatives of which the mathematical elements are known or can be calculated. *The pair of negatives, with a handful of sines, cosines, tangents and other mathematical formulas, is fed into a machine which is one of the supreme wonders of invention. From the two landscape photographs the machine draws a contoured topographic map.*

There is no use trying to explain how this is done. Like the Einstein theory, this machine can be understood by not more than a dozen persons in the world. However, it does the trick with great precision. Such machines have now been developed by at least two European instrument makers. One of them, in Switzerland, though huge prices are charged for its use, is said to be continually swamped with work by railroad builders in various parts of the world. *The railroad surveyor presses the button on the camera; the machine does the rest.* This makes surveying comparatively easy in the most difficult mountain country.

Construction Equipment and Materials

Manufacturer Incorporates Two New Selling Companies

Sale of the products manufactured by the Alexander Milburn Company, Baltimore, Md., will hereafter be made by the Milburn Sales Corporation and the Milburn Paint Spray Corporation, both incorporated on Dec. 31, 1928, under the laws of the State of Maryland. The Alexander Milburn Company was founded in Baltimore more than 21 years ago by Alexander F. Jenkins, and manufactures welding and cutting apparatus, gas regulators, gas generators, portable lights, paint spray guns and guns for greasing purposes, besides other specialties. The Milburn Sales Corporation takes over the selling of all equipment manufactured by the Alexander Milburn Company with the exception of the paint spray equipment and air guns for greasing purposes; these will be handled exclusively by the Milburn Paint Spray Corporation. In connection with this new development in the Alexander Milburn Company, it is also announced that it has paid all of its remaining indebtedness, which comprised current accounts payable only, and started 1929 without obligations of any kind.

New Water Company to Develop Municipal Supplies

One of the major claims of the newly formed Fairbanks Morse Water Supply Company in Chicago is that it eliminates many of the uncertainties in municipal water-supply development. Although its parent organization, Fairbanks Morse & Company, has been a builder of pumps for many years, this new organization approaches water supply from an entirely different angle. After a thorough geological survey and hydraulic investigation of a plant's water supply has shown favorable possibilities, the company agrees to deliver a definite quantity and quality of water at a definite price per thousand gallons. The plan is worked out so that the municipality is not required to invest any money in the pumping plant. Water may be purchased at a fixed price for a period of years, at the end of which time complete title to the plant passes to the city. At any time during the period the municipality also has the option of taking over the plant. The new company is headed by P. D. Bowler as vice-president and general manager. Mr. Bowler is one of the founders of the Layne & Bowler Corporation and was president and general manager of that organization for many years.

Several installations are now under development. At Havre, Mont., a supply of 3,000,000 gal. per day was guaranteed at an investment of $72,000. Three plants are being developed for the borough of Brooklyn, N. Y., to be used for furnishing water for sluicing ashes at incinerator plants and at Muskegon Heights, Mich., a third supply is being developed.

Concrete Pavement Yardage

There is given below a tabulation of concrete pavement yardage awarded during the month of December, 1928, and also the total number of square yards awarded in roads, streets and alleys for the year 1928 up to Dec. 29:

	Sq.Yd. Awarded During December, 1928	Sq.Yd. Awarded From Jan. 1, 1928, to Dec. 29, 1928
Roads.....	5,165,945	93,577,760
Streets.....	2,663,944	50,808,214
Alleys.....	249,696	3,948,452
Total....	8,079,585	148,334,426

The concrete pavement yardage awards made during the month of January, 1929, according to the figures of the Portland Cement Association, are as follows: For roads, 6,088,674 sq.yd.; for streets, 1,308,682 sq.yd., and for alleys, 63,810. The total for the first month of the year is 7,461,166 sq.yd.

Formation of Garage Operating Company Announced

A new company, known as d'Humy Management, Inc., has been formed by Ramp Buildings Corporation, 21 East 40th St., New York City, to operate garage properties, and has taken over eighteen garages in cities east of the Mississippi serving 35,000 customers weekly. It will continue to add to this group, according to the announcement. Clifton D. Jackson, formerly of John Wanamaker, New York, is president, and Clement E. Paxson, president, Ramp Buildings Corporation, is chairman of the board. The parent company, Ramp Buildings Corporation, was formed in 1920, and has been associated since that time with the engineering of 226 garages using the d'Humy system of ramps.

Southeastern Road Show to Be Held in Jacksonville

The Southeastern Road Show to be held in Jacksonville, Fla., March 19-23, by the Florida Road Builders Association in connection with its annual good roads conference will have available 21,000 ft. of exhibit space allocated as follows: Heavy machinery pavilion 14,000 ft.; Conference Hall building, 3,600 ft.; Building No. 1, 4,100 ft. Much of this space has already been contracted for, according to the Florida Road Builders Association, 915 Greenleaf & Crosby Building, Jacksonville, Fla., the officers of which are: President, J. J. Hedrick, president, Florida Paving Contractors Association; vice-president, W. J. Bryson, president, Walter J. Bryson Paving Company; treasurer, Elliott W. Butts, manager, Jacksonville Chamber of Commerce; and secretary, Ike Parrish. Directors are C. C. Brown, Engineering College, University of Florida; Harvey A. Hall, state road department; Charles A. Browne, president, Florida Engineering Society and county engineer of Orange County; Charles Strobel, manager, Georgia Casualty Company; W. E. Sheddan, city engineer, Jacksonville; H. L. McDonald, McDonald & Burgman; Foster Graves, Baker & Holmes.

Business Notes

JEFFREY MANUFACTURING COMPANY, Columbus, Ohio, announces the purchase of the entire common capital stock of the Galion Iron Works & Manufacturing Company, Galion, Ohio, as a part of its plan for additional development into basic industries. The Galion Iron Works & Manufacturing Company is one of the old-established manufacturers of road machinery and in addition has developed elevating and conveying equipment. Units of the latter equipment, which can be economically produced at the Jeffrey plant in Columbus, will add to the Jeffrey output and at the same time give additional capacity at Galion for specialization on road machinery. The new officers of the Galion Company are: D. C. Boyd, chairman of the board; Robert W. Gillespie, president and general manager; J. S. Boyd, vice-president; and F. W. Faber, secretary and treasurer. Mr. Gillespie is also vice-president and general manager of the Jeffrey Manufacturing Company.

BLAW KNOX COMPANY, Pittsburgh, Pa., has purchased the A. W. French Company of Chicago, manufacturer of the Ord concrete finisher and several grading machines.

JAMES P. DALY, who for seven years represented the Texas Company in New Jersey in the sale of asphalt and road oils, has joined the selling organization of the American Tar Products Company, Pittsburgh, Pa., and will represent that company in New Jersey, with headquarters at the Kearny plant.

BRUCE P. SMITH, who has been associated with Western Wheeled Scraper Company and Earth & Rock Equipment Company for a number of years, is now in charge of their Chicago office, located at 105 West Adams St., Bankers Building.

ROGER D. EDWARDS, acting vice-president of the new consolidation between the Ryan Manufacturing Corporation and the C. D. Edwards Manufacturing Company, Albert Lea, Minn., will resign, effective March 1. Mr. Ed-

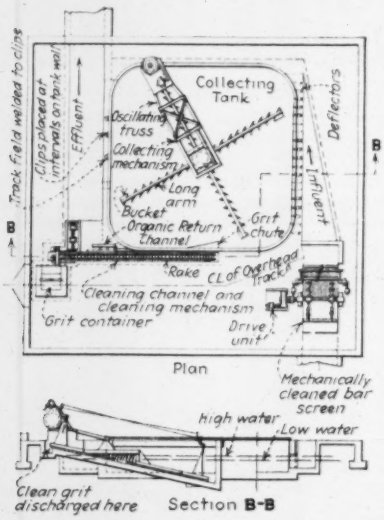
wards had been in active management of the Edwards company for a number of years prior to the sale of his interests to the new organization.

INTERNATIONAL CEMENT CORPORATION, New York City, announces that the names of its domestic subsidiaries are officially changed to the Lone Star Cement Company of the state in which each company is located. The change affects only the name of the company, no change in the management, policies or products being involved.

New Developments

Self-Cleaning Grit Chamber for Sewage Plant

An automatic, self-cleaning grit chamber to be known as the Dorr Detritor, has been introduced by the Dorr Company, New York City, for general use in place of the conventional grit-settling troughs at sewage plants. The name has been taken from the word "detritus," which is usually used to refer to the inorganics in sewage. The machine is said to perform all the functions of the grit-settling channels and in addition

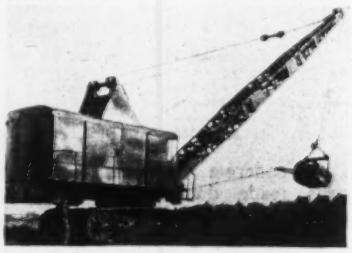


automatically collects and washes the grit, delivering it finally in a clean, dewatered condition to a suitable receptacle. From the accompanying illustration it will be noted that the detritor is located directly after the bar screen and consists essentially of four chief parts—a grit-collecting tank, a collecting mechanism, a grit-cleaning channel and a cleaning mechanism.

The purpose of the square and shallow collecting tank is to reduce the velocity of the incoming sewage to such an extent that practically all of the particles of grit of a predetermined size may settle out within the bounds of the tank. The collecting mechanism is a modification of the Dorr traction clarifier. The plow blades move the grit outward toward the tank sides, where it is picked up by the buckets on the ends of the radial arms and deposited in the grit chute, which discharges the grit into the narrow, rectangular inclined cleaning channel. The cleaning mechanism has a reciprocating motion, the rakes just clearing the inclined bottom and not only advancing the grit toward the point of discharge but also imparting to it a turning over action which releases entrained organic matter. This released organic matter, which is kept in suspension by the gentle selective agitation of the cleaning mechanism, flows back into the collecting tank through an organic return channel in the dividing wall. The washed grit is discharged from the inclined cleaning channel in a dewatered, odorless condition into a grit container.

New Diesel Excavator in 3½-Cu.Yd. Size

A new excavator of 3½-cu.yd. capacity has been added to the line of machines manufactured by the Harnischfeger

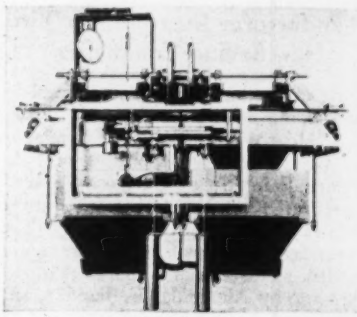


Sales Corporation, Milwaukee, Wis. It is powered by a 175-hp. six-cylinder diesel engine, although it is available with gasoline or electric drive. It is also convertible from shovel to dragline and to crawler crane. It is known as Model 900 and its design closely resembles that of the four smaller cast-steel machines built by this manufacturer. The revolving frame, car body and crawler frames are single piece annealed steel castings, fully machined at the bearing points. The revolving frame weighs 17,000 lb. and its over-all length is 19½ ft.

Roller bearings are used throughout the main machinery of the upper structure, and the four double sets of swiveled, conical steel swing rollers, which carry the weight of the revolving structure, are also provided with anti-friction roller bearings. A box girder shovel boom with outside dipper sticks is used. All gears of the traction mechanism are completely inclosed in heavy cast-steel gear cases and run in a bath of oil. Bearings in the lower structure are provided with renewable bronze bushings. Steering brakes are provided, and the crawlers are gear driven. The over-all width of the machine is 14½ ft. and the over-all length 25 ft.

Proportioning Aggregates by Weight or Volume

Measuring aggregates for concrete on the basis of either weight or volume is possible with the new combination volume-weighing "AggreMeter" of the Erie Steel Construction Company, Erie, Pa. This can be attached not only to all types of bins made by the company but also to bins of other manufacture. It is shipped complete, no assembly being necessary in the field. Capacities of the boxes by volume are: Sand, 9 to 19¼ cu.ft.; stone, 10.2 to 29 cu.ft. The usual scale capacity is 5,000 lb., but



larger scales can be supplied when required.

Only one operator is necessary for the plant, and he is so placed that all operating members are directly visible. The rack-and-pinion gates for filling the boxes may be operated separately or together. Automatic draw-off doors are connected in a similar manner, and close automatically when the material is dumped.

New Variable-Speed Engine for Power Shovel

The new accelerator-controlled power plant used on the 1929 crawler excavator equipment of the Northwest Engineering Company, Chicago, is a distinct departure from the more usual fixed-speed engines. The difficulty in securing proper carburetion and gas distribution for all speeds and all loads in a machine such as a power excavator is claimed to have previously made it impracticable to use variable-speed engines for its type of work. On this new power plant special manifolding and carbureting devices have been installed which give the engine an effective pull at any speed from 450 to 1,200 r.p.m.; in addition it is claimed that the power plant has the ability to respond to any sudden change of speed.

Full manual control of the engine is obtained by movement of a lever which is a hinged portion of one of the regular control levers so that the operator need not remove his hand from the control in order to vary the engine speed. It is claimed that on tests a shovel using this new power plant has shown a 15 to 20 per cent increase in yardage output when working as a variable-speed unit over that when operating at a fixed speed. One of the principal advantages of the variable-speed engine is that it may be opened up to top speed on swinging or hoisting operations, thus simulating steam machinery. Another important advantage comes from the ability to slow down the engine when extremely sensitive handling is necessary, such as setting steel pipe, or on clamshell work where slow speed is desired to place the bucket accurately or to close it easily.